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**THREE CASE HISTORIES OF CRACKING PROBLEMS
ASSOCIATED WITH
STEEL BRIDGE FLOOR BEAMS**

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By
W. H. Walker
M. D. Holbrook
and
P. A. Cassity

A Report on a Research Project
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16. Abstract <p>Floor system cracking problems, the focus of the present study, occur in both structures of recent construction and in older bridges which have been in service for many years. The present study has risen out of specific cases of floor system cracks which are not immediately detrimental to the structural integrity of the floor system and which do not at the outset seem related to a usual design basis load effects. The cases forming the central focus of this report are: (1) The I-474 Shade-Lohmann Bridge over the Illinois River south of Peoria, (2) The I-74 Bridge over the Vermilion River at Danville, and (3) The I-74 Bridge over the Sangamon near Mahomet in Champaign County.</p> <p>The following is seen: Case 1: Cracking due to forces not associated with vertical vehicle loading. That is, evidence is strong that cracking arises from longitudinal load transmitted through the out of plane flexure of the floor beam web in the segment between the connection clip angles and the flange at either end. Calculations show that a very modest induced longitudinal deformation is associated with substantial local stresses at the web to flange junction which is the site of the cracking. Case 2: Fatigue failure of a detail with adverse geometry but with forces induced by vehicle loads which clearly account for the damage. A repair detail to reduce member forces has been suggested and is evaluated. Case 3: The development of a fatigue crack at a cope detail associate with a reasonable stress state for the damage observed, but with poor correlation with a limited controlled vehicle test and predictions of bridge behavior using a grid model. Additional field studies with more extensive instrumentation and a more comprehensive analytical model is needed to resolve uncertainties in this case.</p>			
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by

W. H. Walker, M. D. Holbrook and P. A. Cassity

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DISCLAIMER

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CHAPTER 1

INTRODUCTION

1.1 General

Floor system cracking problems, the focus of the present study, occur in both structures of recent construction and in older bridges which have been in service for many years. Thus this report deals with one of the several issues in the management of maintenance and rehabilitation of bridges in the highway transportation system of the State of Illinois and the nation. Recent inspections of steel bridge floor systems in Illinois have revealed the existence of cracks in floor beam connection and cope regions which are not always attributable to expected, load related, causes. Structural behavior due to loads or effects outside of the usual realm of design calculation is present and deserves study.

The maintenance, rehabilitation and the planning for the replacement of bridge structures consumes an ever increasing portion of resources and of the time of the staff of IDOT and the various state bridge departments. Systematic inspections of bridge structures are now either mandated or attempted as forces permit depending on the class of highway or street usage and ownership. Invariably difficult decisions must be reached where damage is evident and remaining life may be threatened.

The discovery of cracking in steel structures mandates a decision; the crack damage should be measured, documented and carefully studied for at least two structural possibilities:

- The crack is visible and unsightly, but is stable and not growing (i.e., not potentially fatal, nor in a fracture critical member), and action is warranted to remove the crack or treat it so as to preclude re-initiation of propagation so as to eliminate the need for continued and frequent monitoring for possible growth.
- The crack is growing and potentially critical. It may extend to such a length as to become unstable -- the occurrence of a so-called brittle fracture.

The issues associated with the existence of cracks in floor systems are a subset of more general problems associated with the scheduling of maintenance, rehabilitation and the planning for the replacement of bridge structures. The present study has risen out of specific cases of floor system cracks which are not immediately detrimental to the structural integrity of the floor system and which do not at the outset seem related to a usual design basis load

effects. These detected areas of damage represent obvious and visible cracking but are not in the usual sense an issue for remaining overall structural fatigue life. The cracks may be the result of high-cycle fatigue damage or they may be due to other conditions not usually associated with fatigue. It is important to assess whether the cracks found can continue to grow and if they do, at what degree of extension do they represent a significant risk for catastrophic failure.

The existence of a crack represents the completion of the initiation phase of the fatigue process and that the propagation phase is underway. In complex structural assemblies, particularly in welded construction, it is often true that micro-defects and micro-cracks arising from residual stresses pre-exist in the structure before service loading begins. Essentially the fatigue life of the structure in the context of the present study nearly always involves primarily the propagation phase of fatigue life.

The design process, particularly using the common standards of structural modelling and analysis, does not provide design or review information specific enough to address the issue of possible crack propagation in the locations represented in this study as a design limit state. Analysis tools of a higher complexity to provide a three-dimensional modelling of the primary load carrying system, girders or trusses and the floor system are needed as a minimum. This is not to be taken as a criticism of current design practice since the usual approach is historically satisfactory when good detailing based on adequate field experience is invoked.

1.2 Goals of This Report

This study is not intended to modify the current fatigue design procedures for main load carrying members in bridges [1.2]*. The present results could pertain to a structure which has been adequately designed for fatigue but has now developed secondary cracking in the floor system because of phenomena the same as or similar to those discussed herein.

It is the intent that this report both summarize the results of the Project IHR-312 investigation of the subject problem, to serve as a guide to study of the present class of problems, and as an exposition on crack propagation problems at other locations in steel bridge structures. The theoretical background for understanding and predicting the propagation of cracks under cyclic stress applications is well documented in the literature and verified by experimental evidence. This knowledge must be combined with information on site specific loading conditions, the structural behavior both on a gross level and in detail at the critical "hot spot" where cracking has been detected. Case histories will be used to

* Numbers in Brackets refer to items in REFERENCES

illustrate both the results of findings for those cases and as a means for illustrating the investigative approach which is needed in a more general sense.

These methods are useful for review and decision making for repair, major rehabilitation or replacement. It is not intended that these methods be appended to the current fatigue design practice; it would be prohibitive in engineering time to investigate all possible postulated cracking sites for their initiation and propagation life times. Good design practice which minimizes adverse geometry in details and adequate quality assurance for connections is an important factor in "crack free" steel construction.

A significant benefit is associated with a more complete understanding and resolution of the specific floor beam cracking problems presented herein. These are problems which at present are deemed repairable, but which may have a high likelihood of reoccurrence. Without an understanding of the structural phenomena involved, the best present course of action is to repair the floor beam damage with frequent re-inspection of the repair detail and adjacent locations which might be subject to similar cracking. A methodology for handling similar cracking and behavior problems can be directed along the lines followed in this study.

The essential needs in the study involve not only analysis and description of the structure but also a careful assessment of the loading conditions both in magnitude, and traffic volume.

For the subject bridges, available field observations, stress measurements and load studies have been used as background for the present effort. Some general structural analysis of the bridges using 3-D analysis for forces in the region of the cracking zone were made prior to the present study.

All analytical studies have been made with FINITE [8], a comprehensive structural software package with linear and non-linear FEM capabilities, implemented on the Civil Engineering Department computer laboratory facilities. Any comparable structural software product would be equally useful.

1.3 Case Study Approach

Floorbeam cracking problems can not be studied in the abstract; each instance of damage has circumstances of loading, structural restraint, detail geometry, repair potential, etc., that must be taken in context. Hence this problem area is well suited to investigation by the case study approach. But for each case history there are common tasks:

- Identify and quantify the live load induced stresses or other environmentally induced conditions which are capable of initiating and propagating the cracks observed.
- Determine the crack propagation rate and the nature and extent of the fatigue problem present, making use of stress analysis, truck traffic data and load histories as available and fracture and fatigue theory.
- Develop a scheme for remedial action in the present structures and suggest design changes for the future. Verify the suitability of this scheme making use of the above results.
- Apply a methodology for estimating remaining life of a detail to determine when replacement (as an alternative to repair) will become essential. While the above objectives are stated with reference to specific cracking problems in floor beams, these objectives would also bear upon on the problems of fatigue cracking in steel bridge members, i.e., webs, stiffeners, cross frames, etc.

1.4 Case Studies Used

The cases forming the central focus of this report are: (1) The I-474 Shade-Lohmann Bridge over the Illinois River south of Peoria., (2) the I-74 Bridge over the Vermillion River at Danville, and (3) The I-74 Bridge over the Sangamon near Mahomet in Champaign County. These bridges had been the subject of concern of the Bridge Investigation Unit of the Bureau of Materials and Physical Research and the Bureau of Bridges and Structures (IDOT). The studies [7,11,12] were the responsibility of the late F. K. Jacobsen with J. M. South and Ashraf Ali.

The floor system crack damage situation of concern at the outset of this study can be summarized as follows:

(1) In the I-474 Shade-Lohmann bridge the cracks occur at the ends of the floor beam at the junction of the web and the top flange. This location carries zero calculated tension in this crack region. No appreciable stress can be measured due to live load. In addition there are small diagonal cracks at the same location near the clip angles which connect the floor

beams to the main truss system. The bridge is a large three span continuous through truss. The web cracks which are parallel to the flange have been initially arrested with a drilled hole. Re-initiation is an issue. It appears that the cracks may turn downward and grow into the web if not arrested.

(2) At the I-74 Bridge over the Vermillion River cracks were found the the cross-frame connection details. The number of cracks was extensive and a specific retrofit detail for the repair was developed.

(3) At Mahomet the cracks are found in a cope detail at the ends of the floor beam about 20 ft. from the abutment in the side span of a three span continuous structure. The cracks have been arrested by a hole; the issue of re-initiation remains. Measured stresses at the location are significant, probably greater than 12 ksi, under truck traffic. However these larger tensile stresses in the near vertical direction are associated with the vehicle crossing in the center span, not when the vehicle is over the floor beam in question. A structural action involving live load uplift forces seems to be involved.

These cases can be categorized to the extent that three broad situations are covered: cracking where a potential fatigue situation exists; cracking in a region of no apparent fatigue potential; and cracking in secondary bracing where fatigue design is not a usual practice.

1.5 Report Organization

The use of linear elastic fracture mechanics to assess the potential for crack growth is basic to the studies herein and is reviewed in Chapter 2. The three case studies are presented in detail in Chapters 3, 4 and 5. A presentation of a general methodology of approaching other cases of this class is outlined and discussed in Chapter 6; the chapter includes a summary and conclusions.

CHAPTER 2

THEORY OF CRACK INITIATION AND PROPAGATION

2.1 Introduction to Elastic Theory

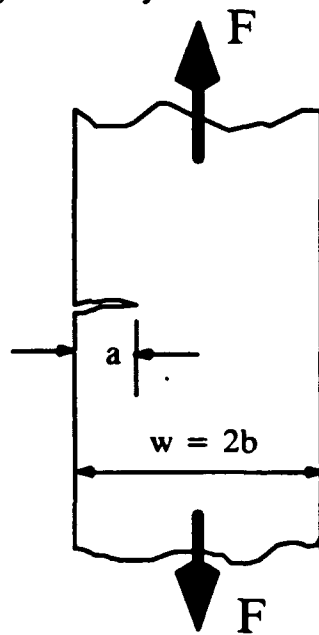
The theory of crack initiation and the linear elastic theory of crack propagation in structural steel is well developed and has been extensively validated in the laboratory and the field [3,4,5,6]. It is useful to review this theory to gain an insight into the significant parameters of the problem.

Primary interest is in the propagation phase of the life of the structural element since with the discovery of visible cracks the initiation phase is complete. That is, the total life, N_T , is the sum of the initiation life, N_I , and the propagation life, N_p . It can be shown that indeed the initiation phase may dominate the total life of the structural element or detail where cracking is present. But, for the present we seek to describe the remaining life of the structural element or detail, i.e., the balance of the stable propagation phase. Alternatively we can extend our analysis to include the effect of introducing hole to arrest the crack or other modification proposed for repair. Lastly we may have to deal with the issue of re-initiation of the crack in the repaired structure.

In thin plate elements typical of the problems studies in this program it is useful to review and discuss first the growth of an edge crack in a moderately wide plate under uniform stress, one of many standard cases that are documented in the literature (Mode I behavior, see Fig. 2.5). The crack is assumed to be of length a and subjected to alternating cycles of stress $\Delta\sigma$ which are, of course, intensified at the crack tip so that the crack growth is a function of the variation of the stress intensity, ΔK . The stress intensity is also a function of a geometry parameter $k(a/b)$. These relationships are summarized in Fig. 2.1.

It is possible to trace the propagation of a crack through a relatively complex structural details by making use of one or a combination of several simple propagation models. Thus the following cases are useful for understanding the phenomena involved and can be concatenated to give a useful picture of a crack propagation, in stages, first as a simple through crack until it reaches, say, a flange plate where, second, it grows as a "penny shaped" internal crack only finally to emerge again as, third, a through crack which threatens the integrity of a main member.

(a) Geometry:



Uniform Stress: $F = \sigma \cdot w \cdot t$

Paris law:

$$da/dN = A(\Delta K)^m$$

where

$$\Delta K = 1.12 \Delta \sigma \sqrt{\pi a} \cdot k\left(\frac{a}{b}\right)$$

and $k\left(\frac{a}{b}\right)$ is the correction factor shown in (b).

(b) Correction factor for width (Data, see Ref. 3):

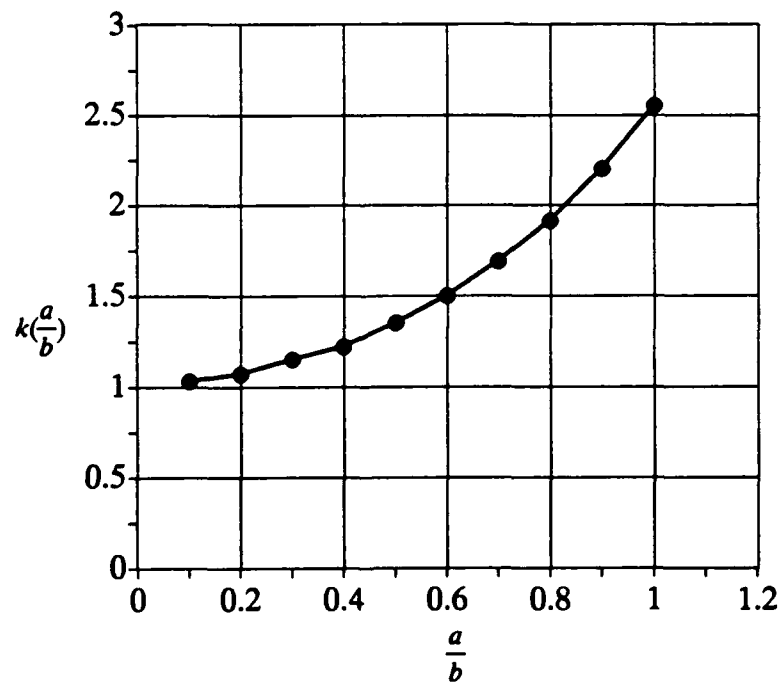
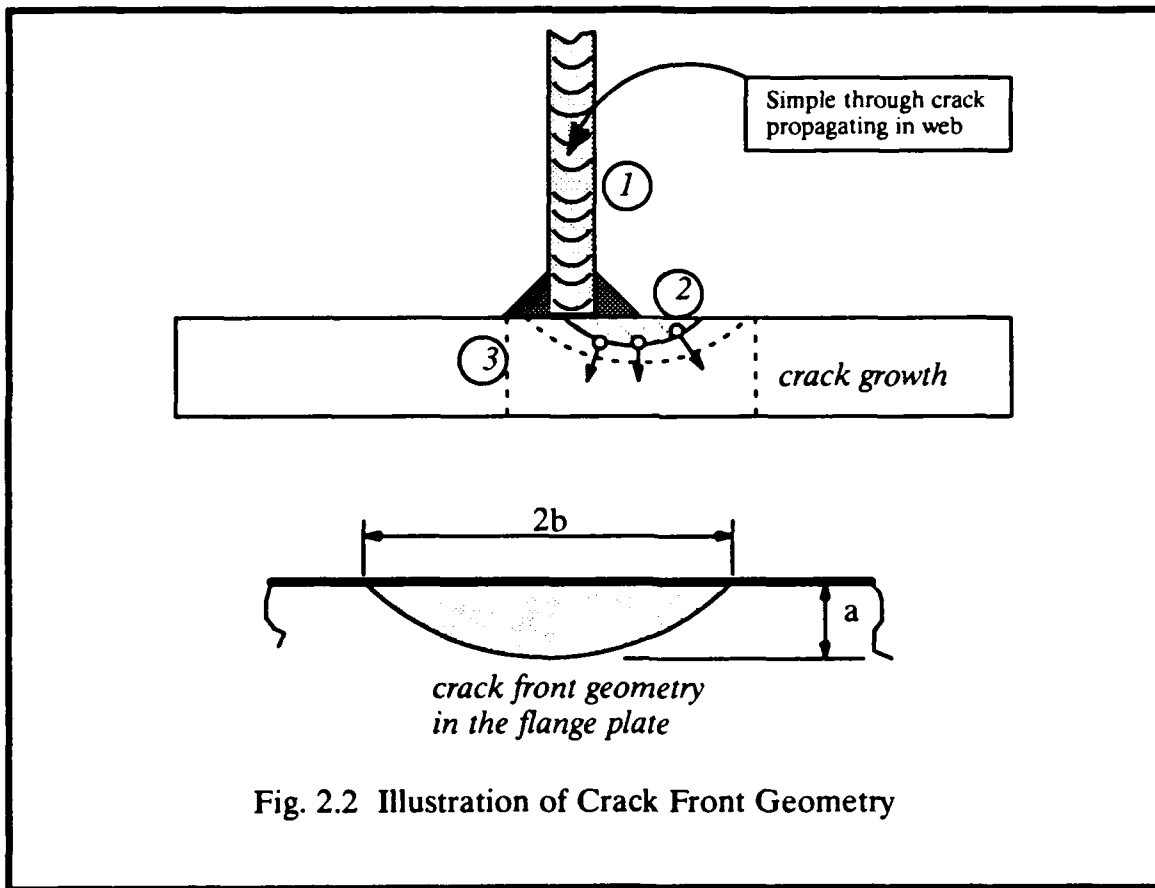


Fig. 2.1 Propagation of a Simple Through Edge Crack

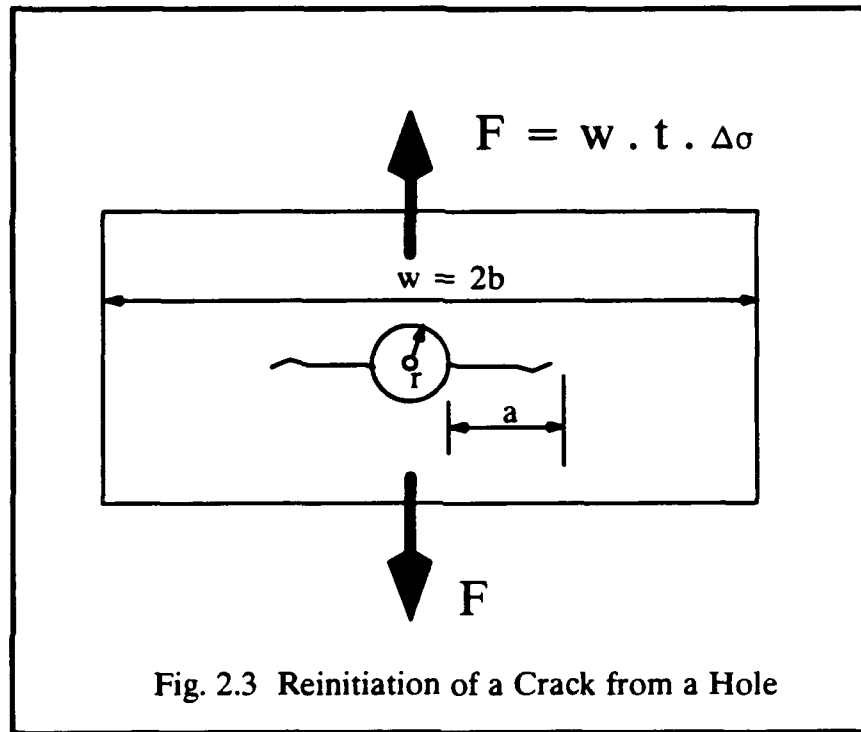
A common result of crack initiation from a welded connection or from a propagating web crack is the initiation of a "penny-shaped" crack in the edge of a flange plate as shown in Fig. 2.2. This a crack advances as an elliptical crack front until it reached the bottom surface of the flange. At that stage it again becomes a through crack and again propagates according to the through crack formulation. The probable mechanism would be propagation into the fillet weld and then down into the flange.



In modelling the propagation of the crack illustrated in Fig. 2.2 it is usual for simplicity to assume a constant proportion for the crack ($\frac{a}{b} = a \text{ constant}$). The actual calculation for ΔK follows the same pattern as for Fig. 2.1, but with additional correction factors[3].

A common remedial measure for a crack is to drill a hole at the crack tip to (1) remove the discontinuity associated with the sharp crack tip and (2) to introduce a known and

moderated stress condition. Thus, it is useful to explore the stress intensity factors for a crack which has re-initiated from a round hole -- a hole which has been drilled to arrest or slow crack growth.



That is, as a through crack formulation as introduced previously corrected for the change in effective stress intensity produced by the hole. The propagation from a hole can be studied using the symmetrical geometry shown in Fig. 2.3. The formulation remains the same with a correction factor $f(\frac{a}{r})$, values of which are plotted in Fig. 2.4.

The more general form of the expression for stress intensity is $\Delta K = Y \Delta\sigma \sqrt{\pi a}$, where Y is a factor which represents the product of all needed correction factors[6,4]. That is, Y will correct for stress gradient, finite width, geometric shape factors, etc. The effect of this factor on predicted life is reflected on page 12 with the modification of the factor C_1 to include a term $(Y)^{-m}$.

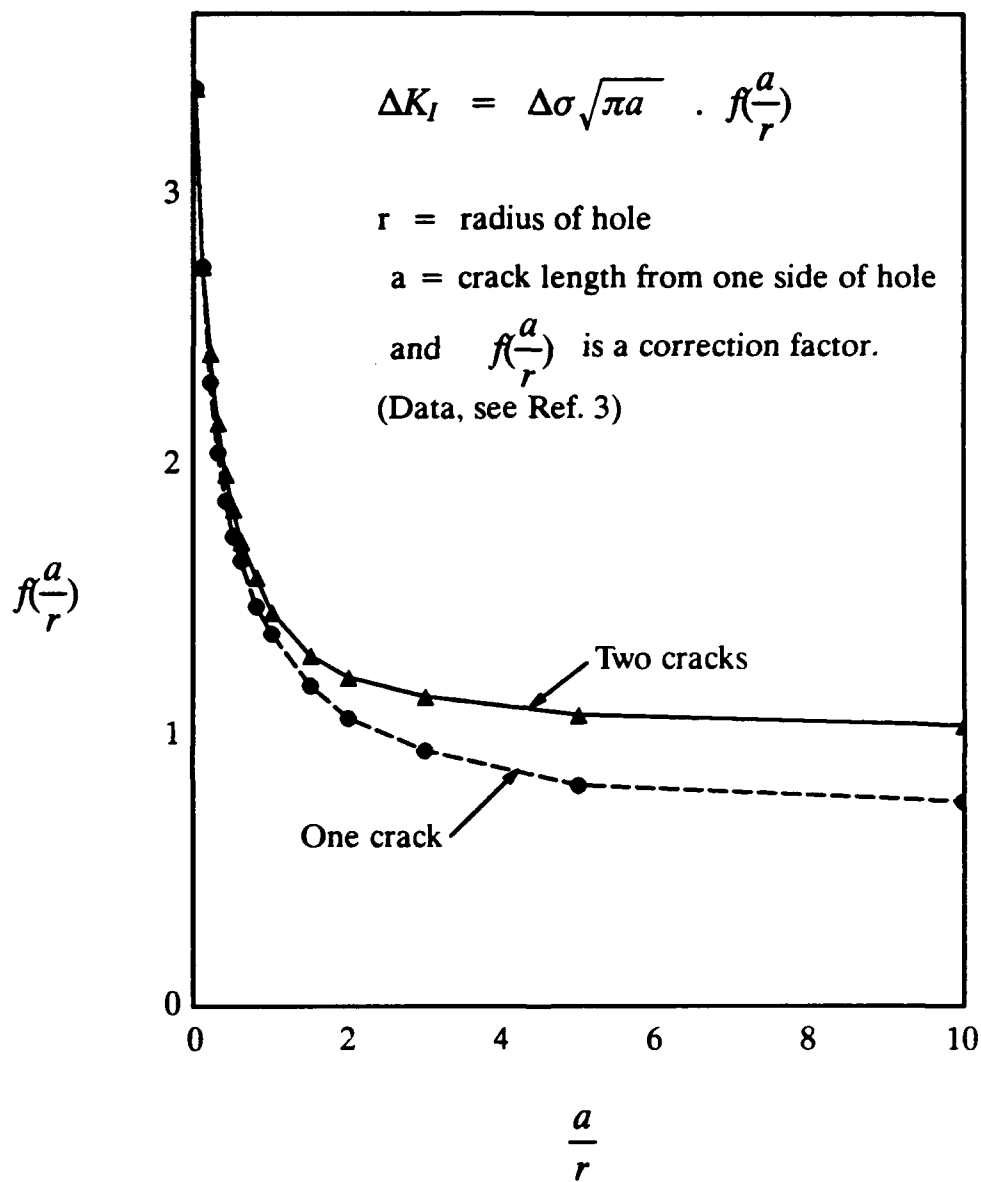
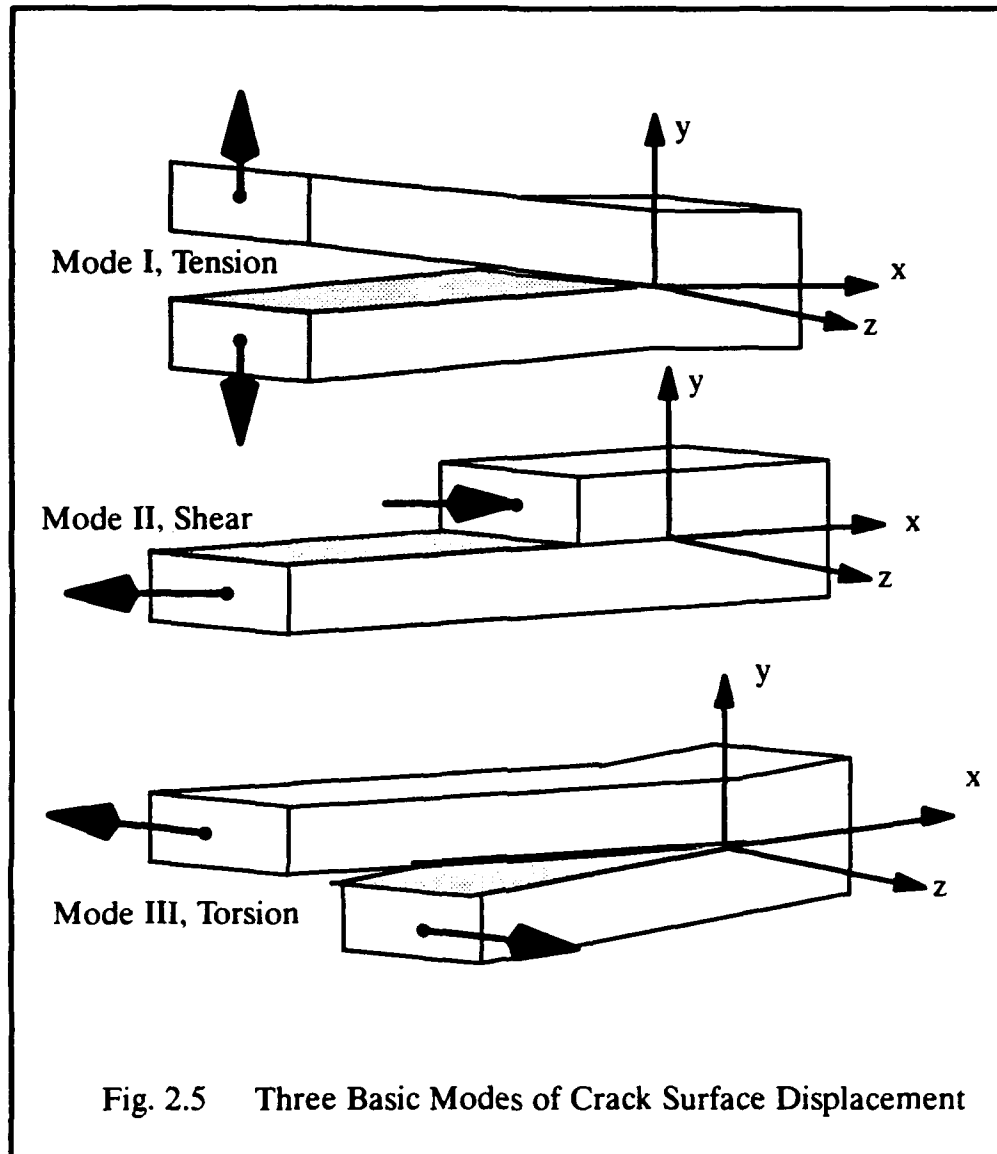


Fig. 2.4 Correction Factor for a Crack Propagating from a Hole



The fracture propagation information discussed has been in the simple crack opening mode wherein the crack is propagating in a tension field -- termed Mode I crack extension as defined in Fig. 2.5. Two other modes are commonly noted: Mode II which is a shearing displacement and Mode III which is a displacement across the crack face out of the plane of the material. Instances exist where mixed modes of behavior are present.

2.2 Crack Propagation Life from an Initial Length

The relationship for da/dN shown in Fig. 2.1 can be recast in a form for the calculation of life (N) corresponding to a growth from an initial crack size to a final crack length considered critical. The parameters A and m in the formulation are material (LEFM) properties which have been established with acceptable reliability for various steels.

Table 2.1 Crack Propagation Characteristics of Various Steels[3]

Steel Type	Typical Structural Grade	A	m
<i>martensitic</i>	<i>ASTM A514</i>	0.66×10^{-8}	2.25
<i>austenitic</i>	<i>AISI 403</i>	3.0×10^{-10}	3.25
<i>ferrite-pearlite</i>	<i>ASTM A36, A572</i>	3.6×10^{-10} **	3.0

** A value of a of 2.0×10^{-10} has been noted by Fisher [4] as average whereas 3.6×10^{-10} is high for structural steels typical of bridge construction; in either case a value of $m = 3$ is appropriate.

Starting with the basic form of the Paris Law [3], $da/dN = A(\Delta K)^m$, for crack growth:

$$dN = \frac{da}{A (\Delta K)^m}$$

$$\Delta K = 1.12 \Delta \sigma \sqrt{\pi a} \quad k\left(\frac{a}{b}\right) = 1$$

$$dN = \frac{da}{A (1.12 \sqrt{\pi})^m (\sqrt{a})^m} \quad \text{and thus,}$$

$$N_p = C_1 \int_{a_i}^{a_f} a^{-\frac{m}{2}} da \quad \text{where,}$$

$$C_1 = \frac{1}{A (1.985)^m (\Delta \sigma)^m}$$

In the formulation for dN above the limits on the integration, initial and final crack length, clearly determine the life to be expected. The initial crack size, a_i , may be specified as a design parameter or may be inferred from studies of laboratory fatigue failures. The final crack size, a_f , must be selected either to represent the largest acceptable crack, non-fatal,

before repair is initiated, or it is limited by the critical crack length for the initiation of an unstable running crack — brittle fracture.

2.3 Critical Crack Length and Brittle Fracture

From the above it is seen that the propagation life calculation may be carried out using a reasonable assumption of initial crack length and a limit on final crack length. The length of existing cracks (when visible) should be measured with reasonable accuracy on the bridge. When cracks are not visible, the value of initial crack length (a_i) may be inferred from test data on similar structural details or assumed on a basis of a length related to the limits of detection. The final crack length corresponding to the propagation life can be interpreted several ways:

- As the maximum length acceptable before repair procedures are deemed essential.
- The length at which the crack will become unstable and a brittle (fast running) crack will result.

The determination of the critical crack length must be related to the material properties which govern fracture sensitivity. There are a number of parameters which are significant including material type, temperature and crack length and geometry.

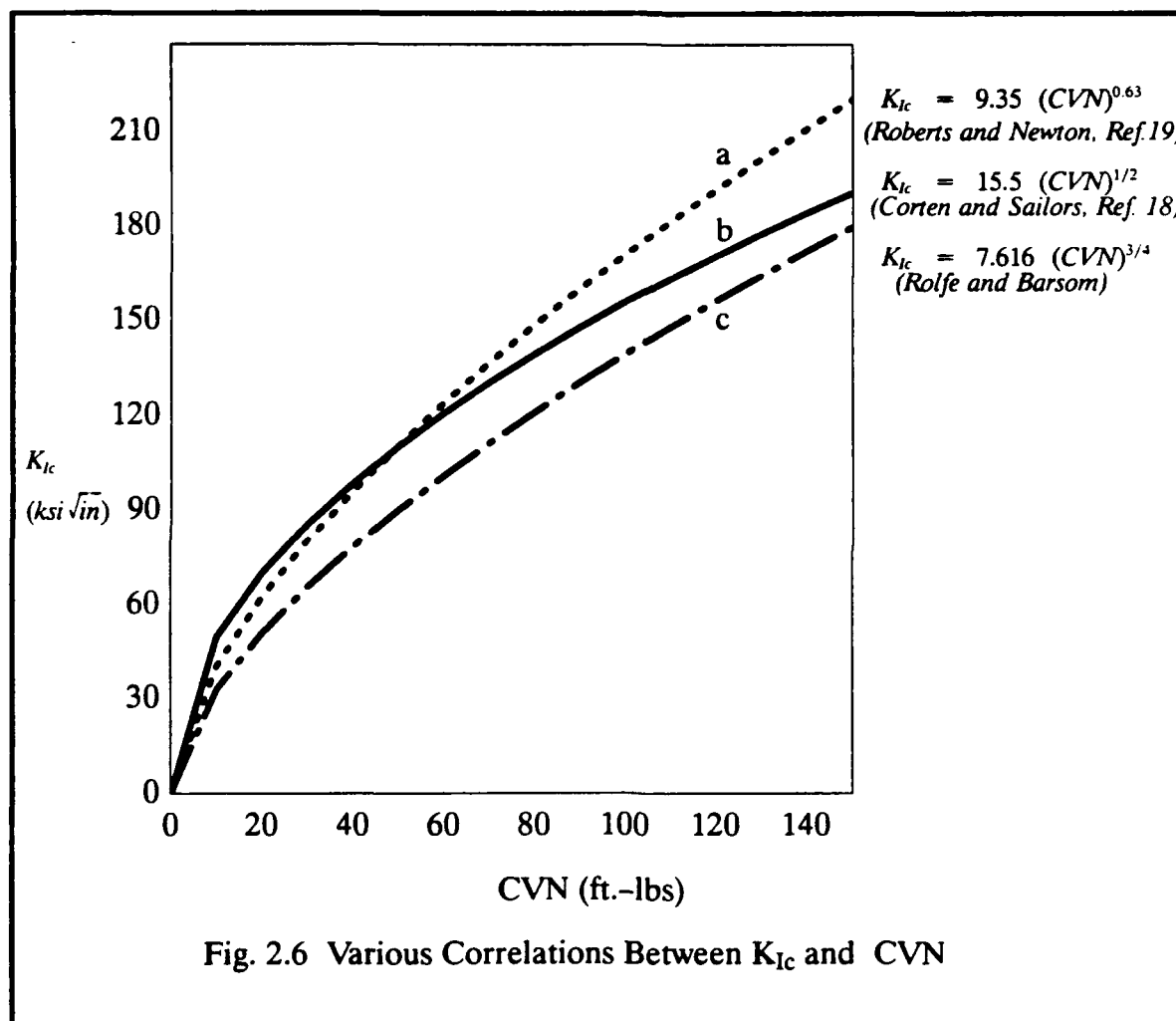
Fracture Toughness, CVN and K_{Ic}

With the existence of a crack determined and the stress intensity at the crack tip estimated, the assessment of unstable behavior requires a determination of the notch toughness of the material at the damage site. Clearly material property determinations including Charpy Tests, other fracture toughness tests such as a three point CTOD test would be desirable, but might not be justified by a low criticality of the crack location or may involve test program costs which exceed a simple repair of the damage. Where testing is not possible the literature provides guidelines which are helpful.

Most toughness measures and the AASHTO specification are presented in terms of limits on Charpy Test results — CVN limits at specified temperatures. The correlation between CVN values and K_{Ic} values is important. One critical element in the determination of K_{Ic} is loading rate. Rolfe and Barsom[3] present several correlations independent of loading rate; numerical values for these are plotted in Fig. 2.6 following to illustrate the scatter in these correlation equations. The most conservative relationship denoted "c", is given as $K_{Ic} = 7.616 (CVN)^{3/4}$. This is derived from the relationship

$$\frac{K_{Ic}^2}{E} = 2 (CVN)^{3/2}. \text{ However this may be quite conservative and is not useful for low}$$

strength steels. A more useful approach is to obtain CVN results and interpret them as dynamic values (K_{Id}) values; the static values are obtained using a temperature shift relationship $T_{\text{shift}} = 215 - 1.5 \sigma_{ys}$. The reliable determination of CVN values for use in linear elastic fracture mechanics (LEFM) work is difficult for low strength steels such as A-36. At the material thickness encountered a large fraction of plastic action is usual and LEFM does not hold. Because of the elastic-plastic action the material undergoes ductile tearing rather than cleavage fracture.



All of the case studies presented herein are for floor systems fabricated of A-36 steel. All probably predate the use of toughness requirements for bridge steels, but A-36 in the thicknesses typical of the three case histories herein has been a fracture tolerant material. Data for K_{Ic} for A-36 steel has been shown to be difficult to evaluate for LEFM [9]; the data presented in Fig. 2.7, reported by Barsom and Rolfe [3], provides a guide to approximate K_{Ic} values consistent with CVN limits. The temperature shift equation is illustrated on this plot.

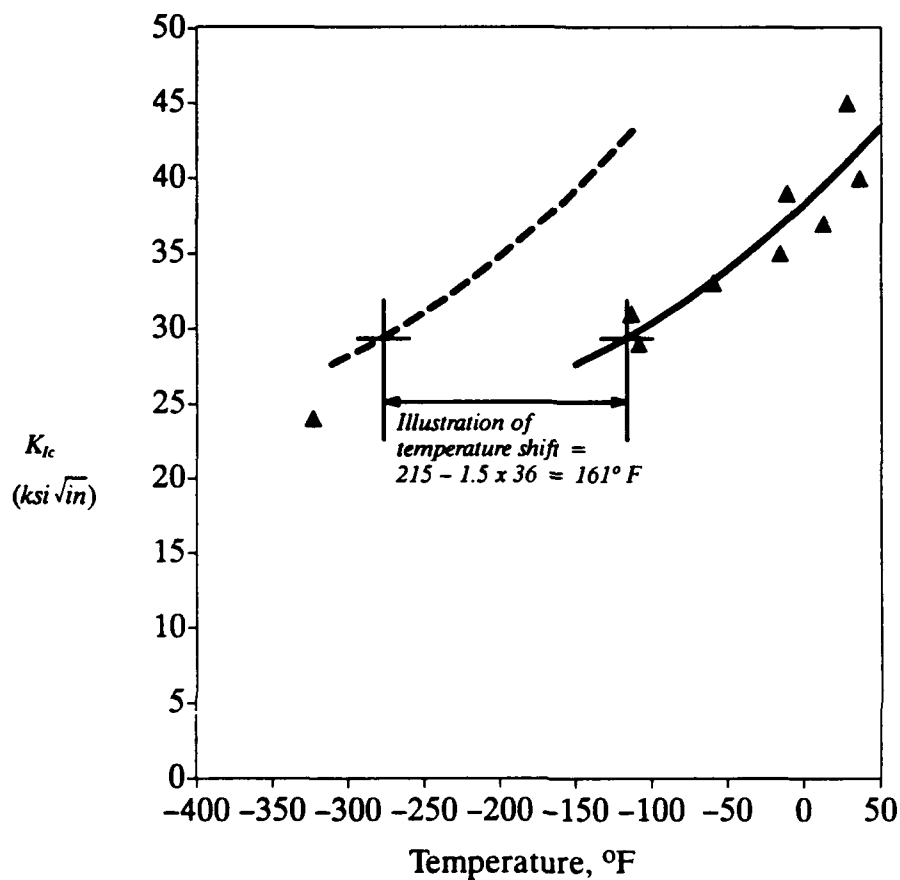


Fig. 2.7 K_{Ic} Data for A-36 Steel Reported by Rolfe and Barsom

The data shown in Fig. 2.7 are for dynamic load, and are terminated at K_{Ic} values for which the parameter controlling plane-strain is exceeded, $\beta = 0.4$. This is the Irwin (LEFM) plane-strain limit, $\beta_{Ic} = \frac{1}{B} (K_{Ic}/\sigma_{ys})^2$, where B is the specimen thickness.

To illustrate the application of a K_{Ic} limit consider a hypothetical case of a location where the cyclic stress induced by traffic is 10 ksi, i.e., $\Delta\sigma = 10$ ksi, and the variation of stress intensity is that of an edge crack on a wide plate element:

(a) $\Delta K = 1.12 \Delta \sigma \sqrt{\pi a}$.

(b) Also let us consider that the temperature and material are consistent with a K_{Ic} value of 40 (ksi $\sqrt{\text{in}}$).

(c) Thus we can write the following relationship: $40 = 1.12 \times 10 \times \sqrt{\pi a_{cr}}$

(d) Solve the expression in (c) for the critical value of crack length, a_{cr} .
Hence, a_{cr} is determined to be 4.1 in.

The determination of a valid level of fracture toughness as measured by K-value (in (ksi $\sqrt{\text{in}}$)) is not simple for A-36 steel for use in limits in LEFM. In the case of material thicknesses from 3/8 up to 3/4 in. seen in the present cases the brittle behavior limit on thickness as given by $\beta_{Ic} = \frac{1}{B} (K_{Ic}/\sigma_{ys})^2$ is violated ($\beta_{Ic} = 0.4$) and ductile tearing can be expected to dominate the failure. That is, for a K_{Ic} value of 40 (ksi $\sqrt{\text{in}}$) and a σ_{yp} of 36 ksi then the value of B calculated is 3.1 in. This value has the meaning that for a material less than say 3 inches in thickness ductile tearing can be expected and LEFM does not hold exactly.

For safety purposes, the lowest value of fracture toughness consistent with the lowest climatic temperature for the bridge's location should be used for any design or repair calculation of critical crack size.

The value of stress variation used above is high for at least one of our cases histories. If the above calculation is repeated for only $\Delta \sigma = 2$ ksi then the calculated value of a_{cr} is determined to be 102 in. Such a value is beyond the geometric validity of the model being used or the dimensions of the structural cross sections of concern herein.

CHAPTER 3

CASE 1: THE SHADE-LOHMANN BRIDGE

3.1 Description of Bridges

The I-474 Shade-Lohmann Bridges are a pair of identical three span, cantilever through trusses, as sketched (not to scale) in Fig. 3.1. These twin truss bridges, built in 1973, carry interstate highway, I-474, over the Illinois River at Creve Coeur in Tazewell County, Illinois. The bridges are designed for HS20-44 live load, and are symmetrical about the center with 300 ft. anchor spans and a 540 ft. main span; the main span contains a 300 ft. long suspended center section. The panel point designation scheme used for identification of crack damage in the floorbeams is illustrated in the sketch of the bridge shown in Fig. 3.1. The two spans are identical and each truss is symmetrical about the centerline. Panel points 14, 15 and 16 are at the hinge section of the suspended center span; the vertical member at panel 15 (U15-L15) is the span hanger. Member L14-L15 and U15-U16 have slotted connections at points L15 and U15 to provide for longitudinal movement. The average daily truck traffic (ADTT) was 1450 in 1984.

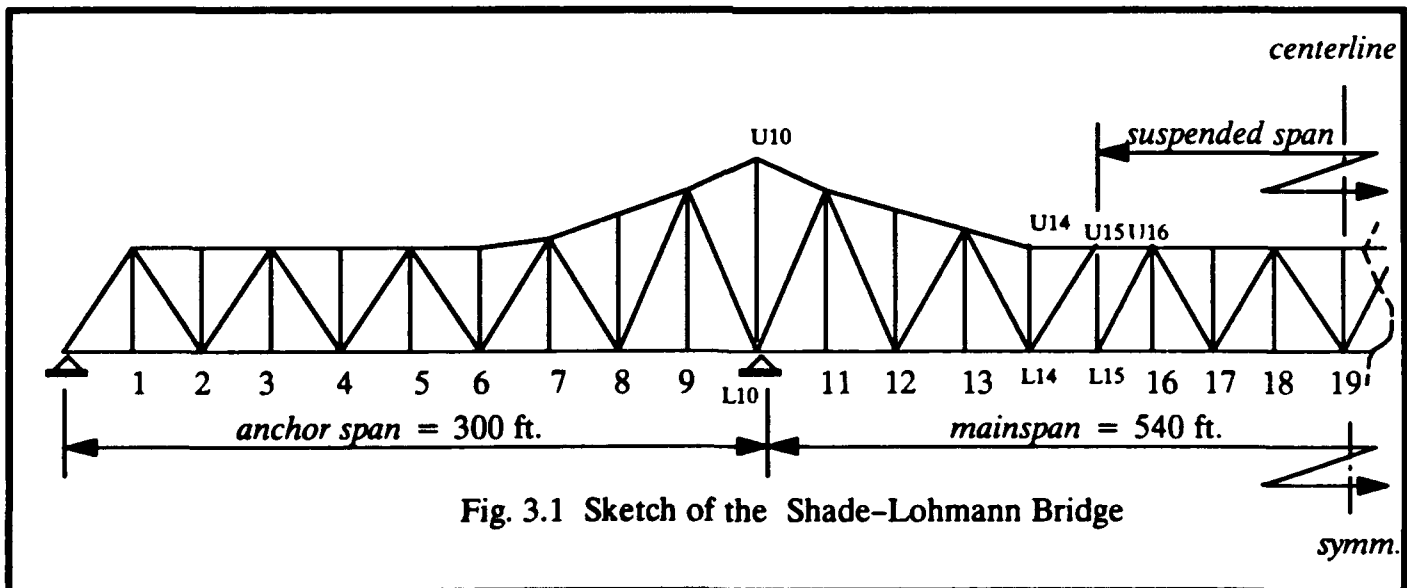


Fig. 3.1 Sketch of the Shade-Lohmann Bridge

The trusses are 42 ft. deep in the parallel chord segments in the anchor and suspended spans and increase in depth to 76 ft. over the interior piers. The composite concrete deck has a

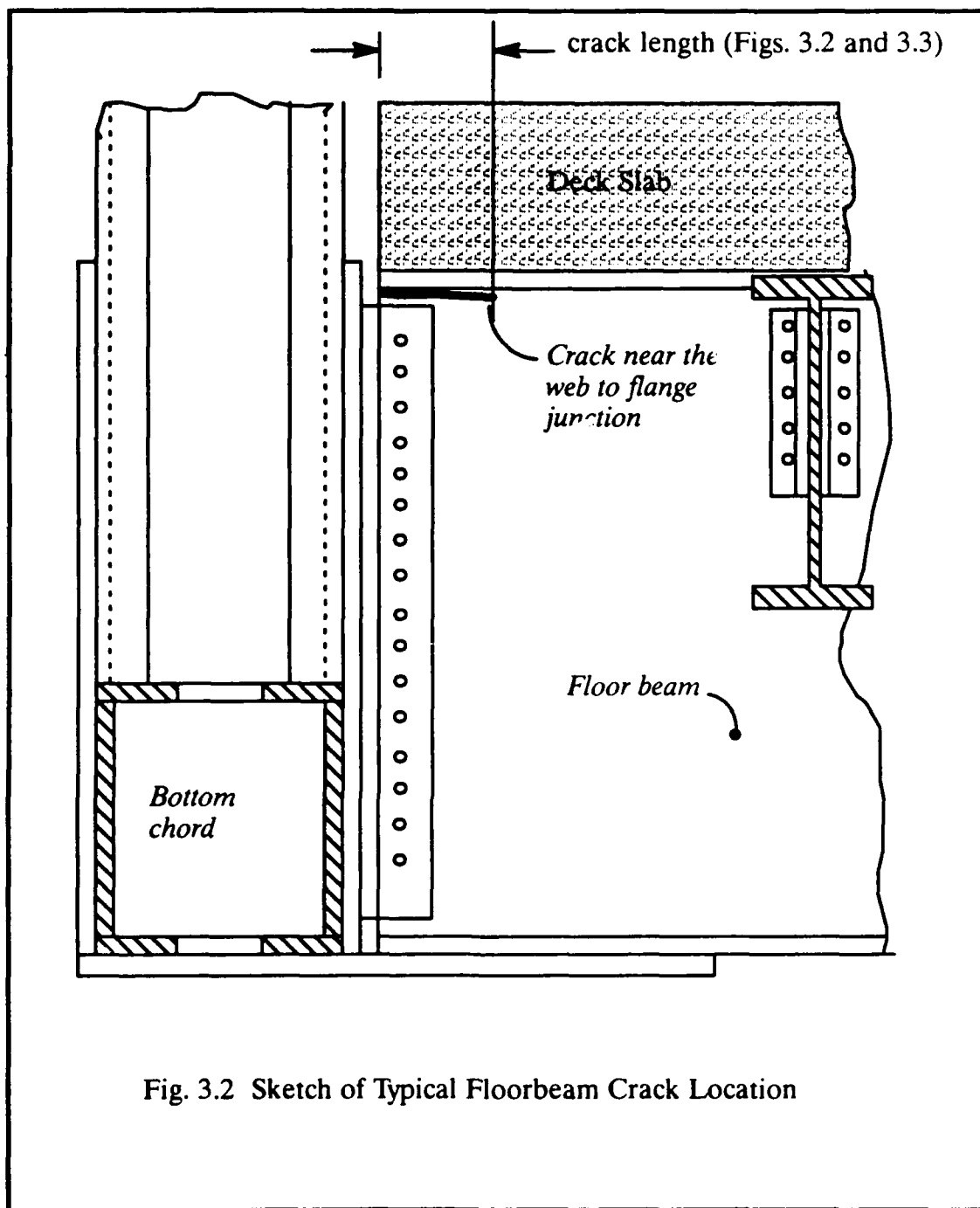
thickness of 7 1/2 inches and a total width of 42.5 feet. The clear roadway width is 39 feet. The deck has been covered with a coal tar interlayer and 1 1/2 inch thick bituminous concrete wearing surface. Both superstructures have welded plate girder approach spans.

3.2 Inspection and Detection

Inspections (1986–1989) of the steel girder floor beams on the Shade–Lohmann Bridges revealed the existence of cracks [11] which are not attributable to expected causes. These cracks occur in the web to flange weld at the ends of the floor beam girders and are propagating horizontally inward (i.e. towards the center-line of the bridge). A typical crack configuration is illustrated in a sketch of the floor beam end with the crack location is shown in Fig. 3.2 and a typical crack is also shown in the photograph in Fig. 3.3.

The cracks typified by the sketch in Fig. 3.2 are located just slightly below the junction of the web and flange of the floor beam at the fillet weld toe. The inspection report [11, p.13] notes the following:

"The predominant and most serious defect occurring in the truss spans is cracking in the ends of the floorbeams. The cracks are in the top flange-to-web fillet weld toe at both ends of most of the floor beams and usually on both sides of the web. Holes were drilled in the webs and through welds at or near the tip of the cracks to arrest crack growth. A few of the cracks, though, have propagated past the drill holes and still others past a second hole drilled because of previous crack growth. New cracks have also been discovered in the floorbeam ends at locations where none were first reported. A few locations were also found where cracks have initiated in the webs where small tack welds were used to secure the clip angles of the floorbeam. .."



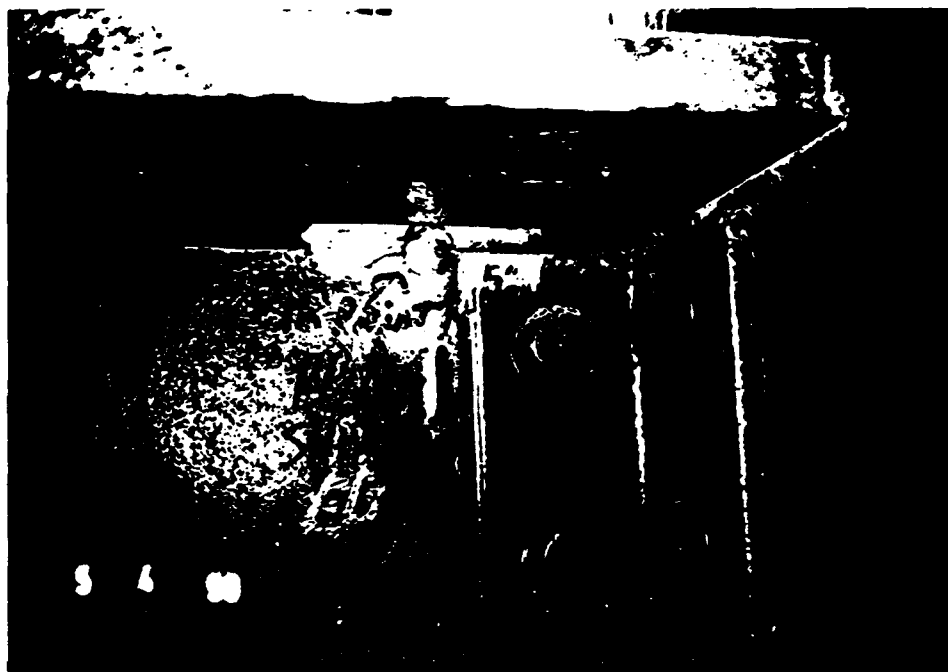


Fig. 3.3 Photograph of Typical Web Cracking in Floor Beam

The occurrence of the cracks in the floor beam ends at the top flange-to-web region at various panel points along the span is illustrated in Fig. 3.4a and 3.4b for half of the span carrying eastbound I-474 traffic. Fig. 3.4a refers to the upstream end of the floor beam and 3.4b to the downstream end. Note that the alignment of the bridge is more nearly north-south as it crosses the Illinois River at this location. These data come from the detailed tabulation [11] of inspection results for 1985 and 1986.

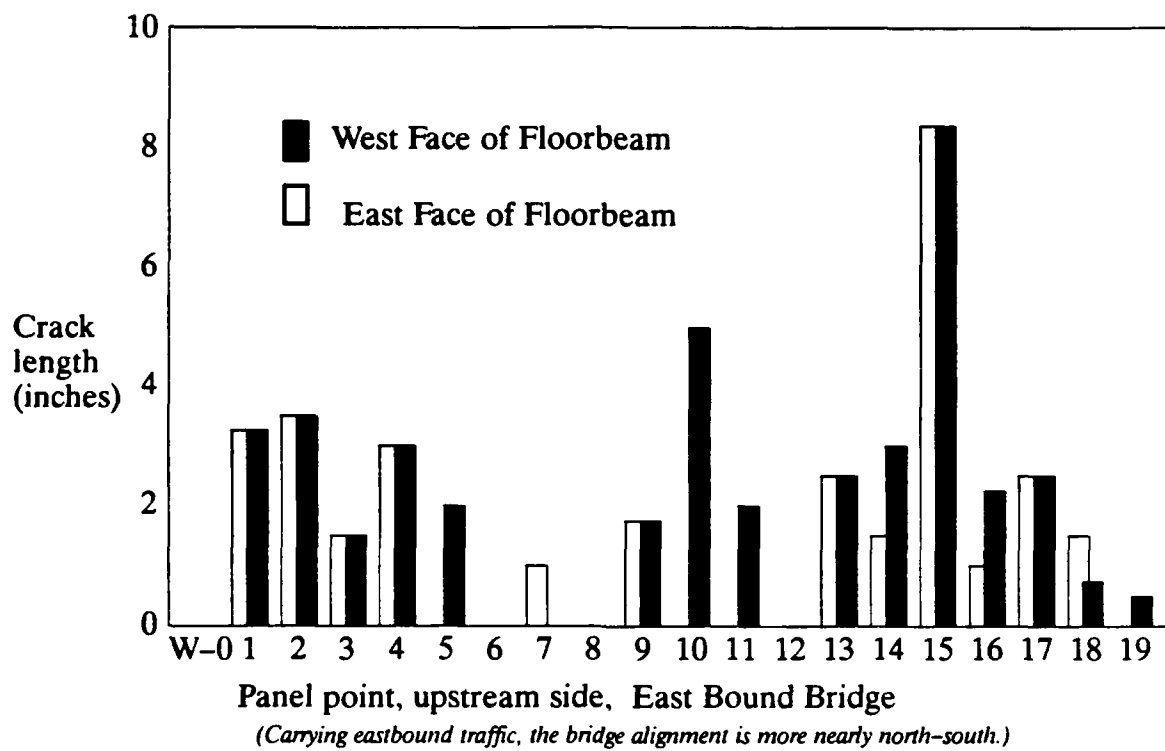


Fig. 3.4a Crack Measurements at Floorbeam Ends, Various Panel Points

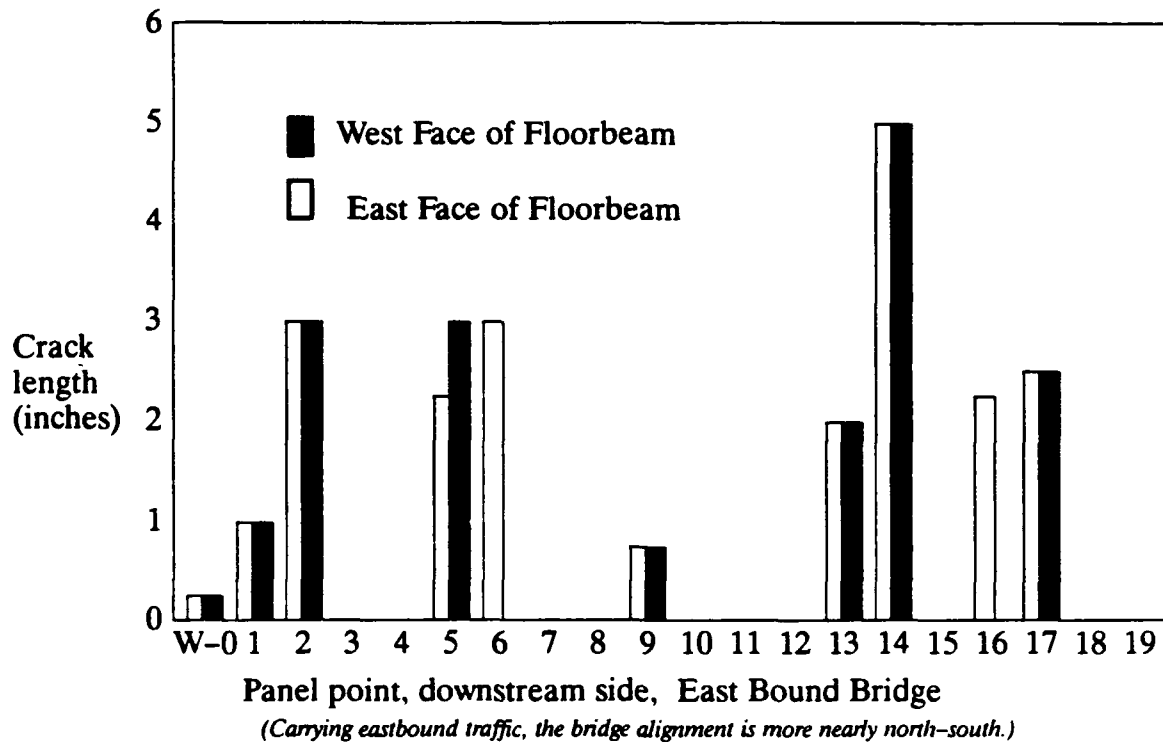


Fig. 3.4b Crack Measurements at Floorbeam Ends, Various Panel Points

The most common crack lengths are on the order of 2 to 3 in., but with instances of locations where cracks of 6 to 8 in. are present.

The above has emphasized sites where cracking occurs. In reviewing all data, the two truss spans have a total of 156 floorbeam end locations; if two faces of the web surface are considered, 312 possible surface crack locations exist to inspect. Of the 312 locations, 71 on the east bound and 57 on the west bound bridges, respectively, were found free of visible cracks or 46 % and 37 %, respectively, free of cracking. In Fig. 3.5a and 3.5b additional measurements are shown for both ends of the floorbeams located at panel points near the suspended span ends in both truss, i.e. carrying west and east bound traffic, respectively.

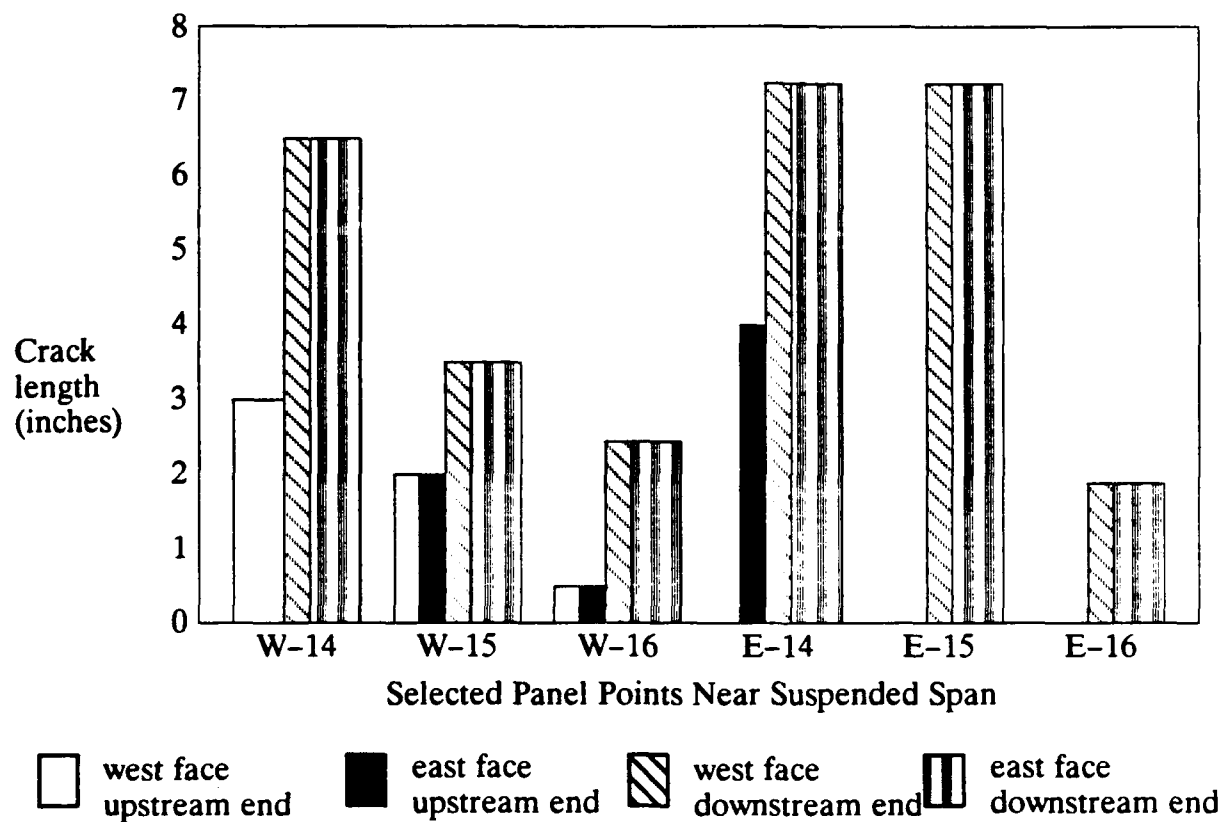


Fig. 3.5a Crack Measurements at Floorbeam Ends Near Suspended Span Hangers
West Bound Bridge

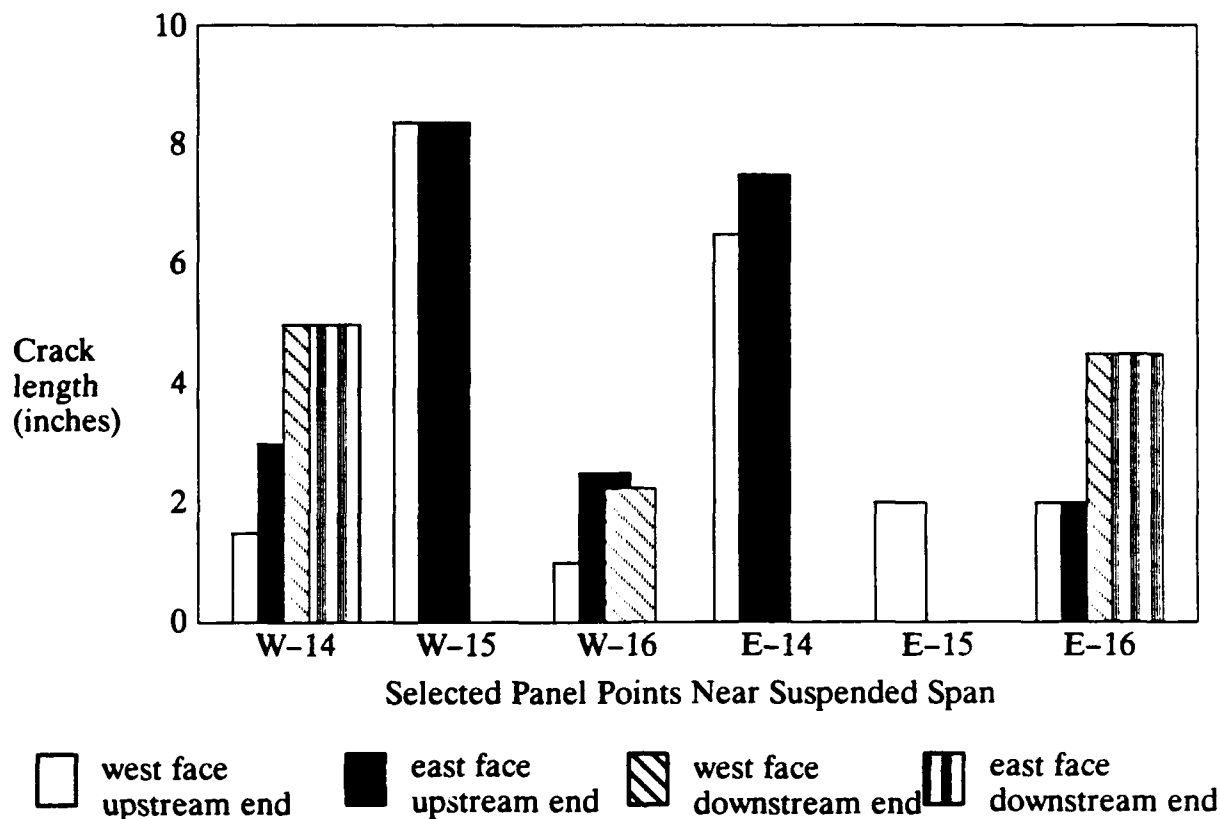


Fig. 3.5b Crack Measurements at Floorbeam Ends Near Suspended Span Hangers
East Bound Bridge

Since the crack locations described above are near the top flange and adjacent to a clip angle connection on the web, it would be usual to assume some bending moment at this point. The degree of restraint offered by the web-only clip-angle connection is less than a full rigid connection assumed in three dimensional frame behavior.

The crack location is assumed in design to be under nominally low or zero calculated bending stress (a pinned end), but the stresses measured in the field are significant. The stresses must be attributable to structural behavior outside the realm of the design assumptions. An attempt to arrest the crack growth has been made by drilling holes at the crack tips of some of the longer cracks. In a few of these cracks with drilled holes there has been re-initiation of the crack out of the drilled hole; see Fig. 3.3.

It appears that the cracks have a tendency to turn downward and grow into the web unless arrested. Growth downward in the web will serve to reduce shear capacity at this location and is of concern. It should be emphasized that this crack growth is in a region of low computed live-load stress.




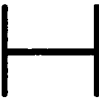


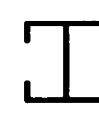

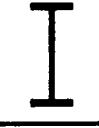
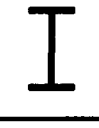
3.3 Finite Element Model

These truss bridges taken with their deck systems represent a potential for a very large, complex analytical model. The floor beam distress was deemed not sensitive to the global modelling of the entire bridge deck slab, although it might be influential in the gross structural action of the entire bridge truss system. The influence of the deck could be reconsidered the the region adjacent to the cracked floor beam ends.

The entire truss bridge was modeled using the FINITE[8], a general structural and finite element analysis program. In Table 3.1 typical member types are shown; these are selected from parts of the structure experiencing more extensive floor beam cracking. The bridge was modeled as both a three-dimensional pin-connected space-truss and a space-frame for comparison of the results. These models comprised 374 members and 152 nodes (joints). For gross structural behavior of the main members the models acted very much the same. The axial loads were identical and the deflections were also the same. The pin-connected truss, of course, did not yield results for so-called secondary moments at the joints.

The three-dimensional space-frame model is essential to accurately model the floor beam connections so as to produce the primary and secondary moments in the floor beam ends at the location of the cracks. The floor beam connection is made to the web of the plate girder floor beam using dual riveted angles with 13 rivets along the angle. In addition, there is a horizontal bracing gusset plate which attaches to the floor beam bottom flange. This detail inhibits the rotation of the end of the floor beams but was not designed as a true moment resistant connection. For analysis, the connections were assumed to behave with full fixed-end connectivity and thus moments are present at the ends of the members. It should be noted that in actuality the top flanges of the floor beams are not connected to the truss and are ineffective in carrying large stresses due to moment at the ends of the floor beam.

The bridge was loaded with the dead load of the steel and nine inch thick concrete deck, and the live load of an HS-20 truck on the floor beams of interest. That is, the effect of a 32 kip axle at standard wheel spacing was shifted transversely across the floor beam to produce an influence line for maximum nominal stress at the weld location. This influence line is shown in Fig. 3.6.

Table 3.1 Selected Member Descriptions for Shade-Lohmann Bridge				
No.	Designation	Sketch	Size	Notes
20	L18-L19		Web: 18 7/8 in.x 1/2 in. Flange: 24 in. x 9/16 in.	Bottom chord near centerline.
57	U18-U19		Web: 24 7/8 in.x 7/8 in. Flange: 20 in. x 1/2 in.	Top chord near centerline. 8x16 in. perf.
28	L10-L11		Web: 24 in.x 2 in. Flange: 20 in. x 1/2 in.	Bottom chord in cantilever arm. 8x16 in. perf.
65	U10-U11		Web: 18 in.x 1 1/8 in. Flange: 23 in. x 1 1/2 in.	Top chord in cantilever arm.
130	U10-L10		Web: 27 in.x 2 1/8 in. Flange: 20 in. x 1 1/8 in.	Vertical post over pier. 8x16 in. perf.
120	U15-L15		Web: 25 in.x 3/4 in. Flange: 18 in. x 1/2 in.	Hanger, suspended span reaction.
118	U16-L16		Web: 20 5/16 in.x 3/8 in. Flange: 12 C 20.7	Hanger in suspended span.
322	L15-L15'		Web: 57 in.x 3/8 in. @end, 61 3/4 in. center. Flange: 18 in. x 1 1/4 in.	Transverse Floorbeam Variable depth.
321	L16-L16'		Web: 57 in.x 3/8 in. @end, 61 3/4 in. center. Flange: 18 in. x 1 1/4 in.	Transverse Floorbeam Variable depth.
359	U17-U17'		Web: 23 1/2 in.x 1/2 in. Flange: 10 in. x 1/2 in.	Transverse top strut

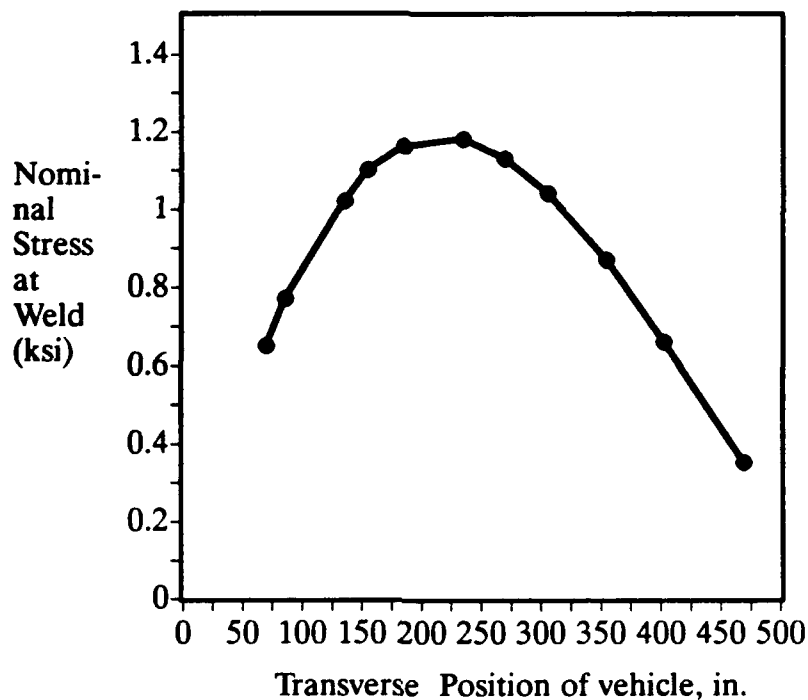


Fig. 3.6 Effect of Transverse Vehicle Position on Stress at End of Floorbeam

The flexural stress range at the crack locations was determined to be about 1.2 ksi maximum, and does not seem to be the cause of the cracking distress. The most important factor being that simple flexural stress of this prediction are parallel rather than perpendicular to the path of travel of the crack. The stress estimate assumes fully rigid connections between the floorbeam and the truss gusset perpendicular to the plane of the truss -- where in fact only shear clip angles on the web of the floorbeam are present.

Neglecting the discrepancy between the direction of the stress and the direction of propagation of the crack, this stress range of 1.2 ksi can be used to perform a fatigue life calculation using a simple edge crack path model. It is assumed that there was a 0.1 inch edge crack present at the time of fabrication and that a final crack length of 3 in. is representative. Thus from Art. 2.2:

$$N_p = C_1 \int_{0.1}^{3in.} a^{-\frac{3}{2}} da, \quad m = 3, \quad A = 2.0 \times 10^{-10} \quad C_1 = \frac{1}{A (1.985)^3 (1.2ksi)^3}$$

$$N_p = C_1 \int_{0.1}^{3in.} a^{-\frac{3}{2}} da = 3.6995 \times 10^8 \left(-2 \left[\frac{1}{\sqrt{0.1}} - \frac{1}{\sqrt{3}} \right] \right) = 9.6 \times 10^8 \text{ cycles}$$

The large number of cycles, 9.6×10^8 , required for propagation of these cracks at a $\Delta\sigma$ of only 1.2 ksi, shows that this level of stress does not account for the damage. Other driving forces must be present.

Some conclusions are evident from the above:

- Floor beams with the worst damage are all adjacent to expansion joints in the bridge deck. This is consistent with the hypothesis that the horizontal cracks at the flange to web weld are due to longitudinal movement of the floor system relative to the bottom chord panel points of the trusses.
- The beams that are near the expansion joints in the pavement have the least amount of restraint against this relative movement and are thus experiencing the greater distress.
- This relative longitudinal movement causes out-of-plane movement of the floor beam web above the connecting clip angles at the web end and induces high stresses adjacent to the weld at the web to flange junction.

The damage discussed above was seen in the case of the floor system of the Prairie Du Chien Tied Arch Bridge studied by Fisher [4] in the which he attributed fatigue damage to longitudinal deck forces acting on a web and flange model.

In addition to the studies just described, the behavior of the bridge model under the action of vertical vehicle loads was explored for various combinations of member release conditions which simulated the supports for the suspended span. The releases include moment release representing the pinned hanger members (such as U15-I15) and the slotted horizontal members (L14-L15 or U15-U16) which permit expansion of the suspended span. In immobilizing these release conditions, to simulate failure of the slots or pins to act, no significant adverse force or distortion conditions were seen at the points of serious cracking.

3.4 Repair and Fracture Control

The action needed to provide remediation for this form of damage is better understood in the context of the sensitivity of the flange and web to longitudinal loads transmitted at the top flange level, for example as sketched in Fig. 3.7. All of the force must be

transmitted in flexure across the segment of web between the clip angles and the top flange -- a distance g on the sketch. For simplicity this web segment is treated as a fixed-fixed beam segment. To illustrate the sensitivity of the detail, a segment of web 12 in. long is considered. If $g = 4$ in. and $t_w = 3/8$ in. then the section modulus is $I = \frac{1}{12} b d^3 = 0.05273 \text{ in.}^4$, and $S = \frac{1}{6} b d^2 = 0.28125 \text{ in.}^3$. If the longitudinal force is taken to be $F_l = 1$ kip, then the calculated stress, σ_b , is 7.1 ksi and the deflection, δ , is only 0.0035 in. In contrast to the analysis on the previous page, if one modifies the calculation of N_p for the a stress range of 7.1 ksi, i.e. $C_1 = \frac{1}{A (1.985)^3 (7.1 \text{ ksi})^3}$, then $N_p = 4.2 \times 10^6$ cycles; if the stress range is as much as 14.2 ksi then only 580,000 cycles are required to propagate a 3 in. long crack. Thus if only a modest fraction of the vehicle longitudinal loads are transmitted in this manner a substantial stress range will be induced at the web-to-flange junction. The relatively small movement associated with a significant stress suggests that this phenomena may not be immediately evident during inspection without measurements in the field.

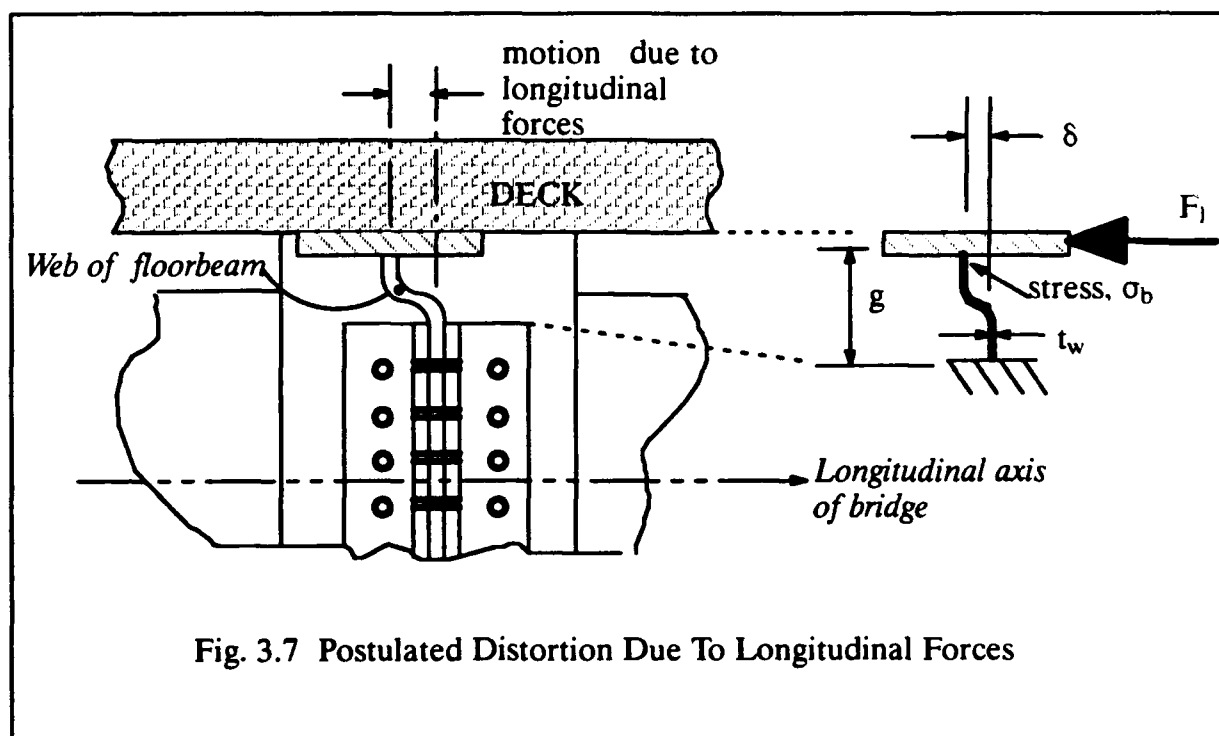
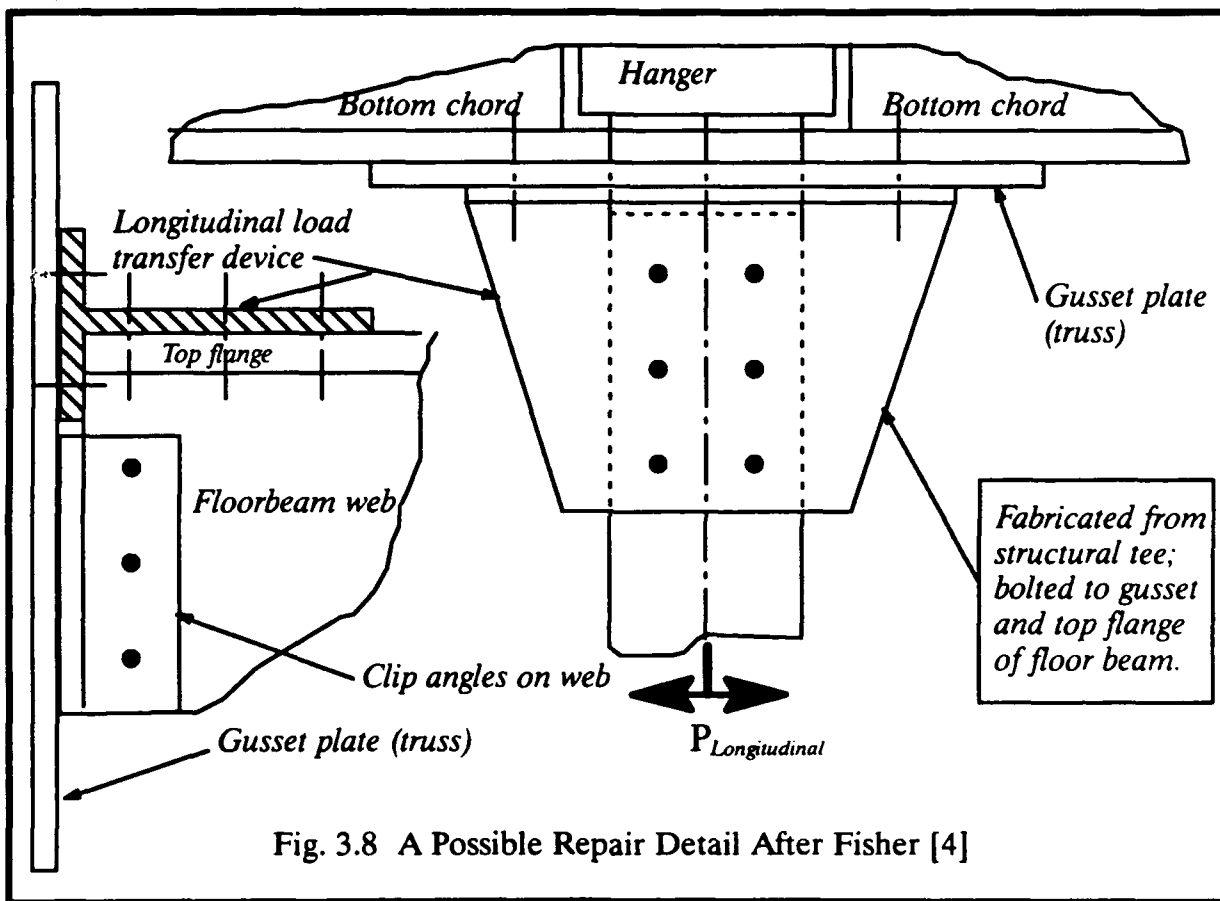


Fig. 3.7 Postulated Distortion Due To Longitudinal Forces

Thus the remedial repair detail for this mechanism of damage must provide a direct load path from the deck and floor beams to the truss at each panel point. A possible repair solution is shown in Fig. 3.8 and is modelled after that described by Fisher in the case cited

above [4]. A connection to prevent the relative longitudinal movement of the floor system must include gussets in the horizontal plane to carry longitudinal loads directly from the floor system to the panel points of the truss. A similar longitudinal load capacity would probably exist if full moment connections were provided at the ends of the floor beams. However, the moment connection between the floor beam and a truss vertical may induce fatigue problems in the truss vertical (hanger) due to secondary bending.



Also, a detail where the slab is separated from the flange at the edge of the girder for some distance, see Fig. 3.9, was included on the drawings of the Shade-Lohmann Bridge, but it was not built as shown, presumably to permit the use of simpler form-work for the full width of the deck. The region of separation would have the possible effect of mitigating the stress concentration at the end of the floor beam to web detail and weld. It might also serve to direct more longitudinal force into the floor stringer system and to reduce the relative stiffness of the structural element at the top flange level carrying force into the clip angles. It is

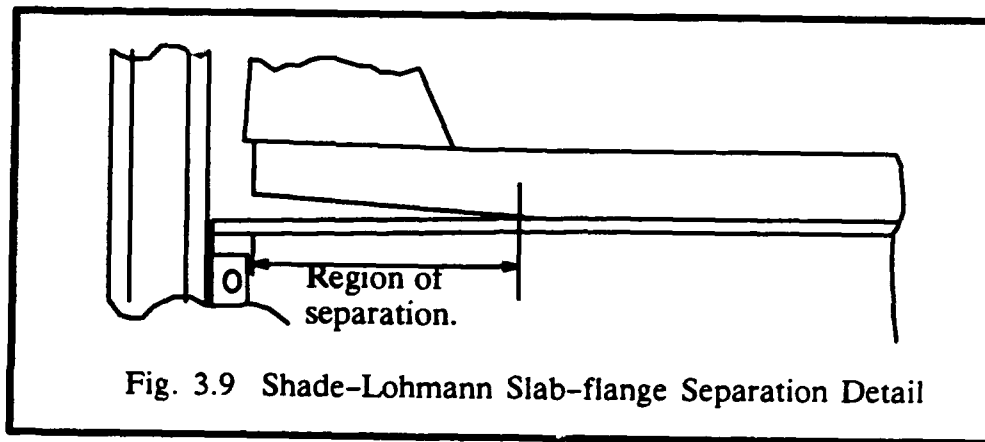


Fig. 3.9 Shade-Lohmann Slab-flange Separation Detail

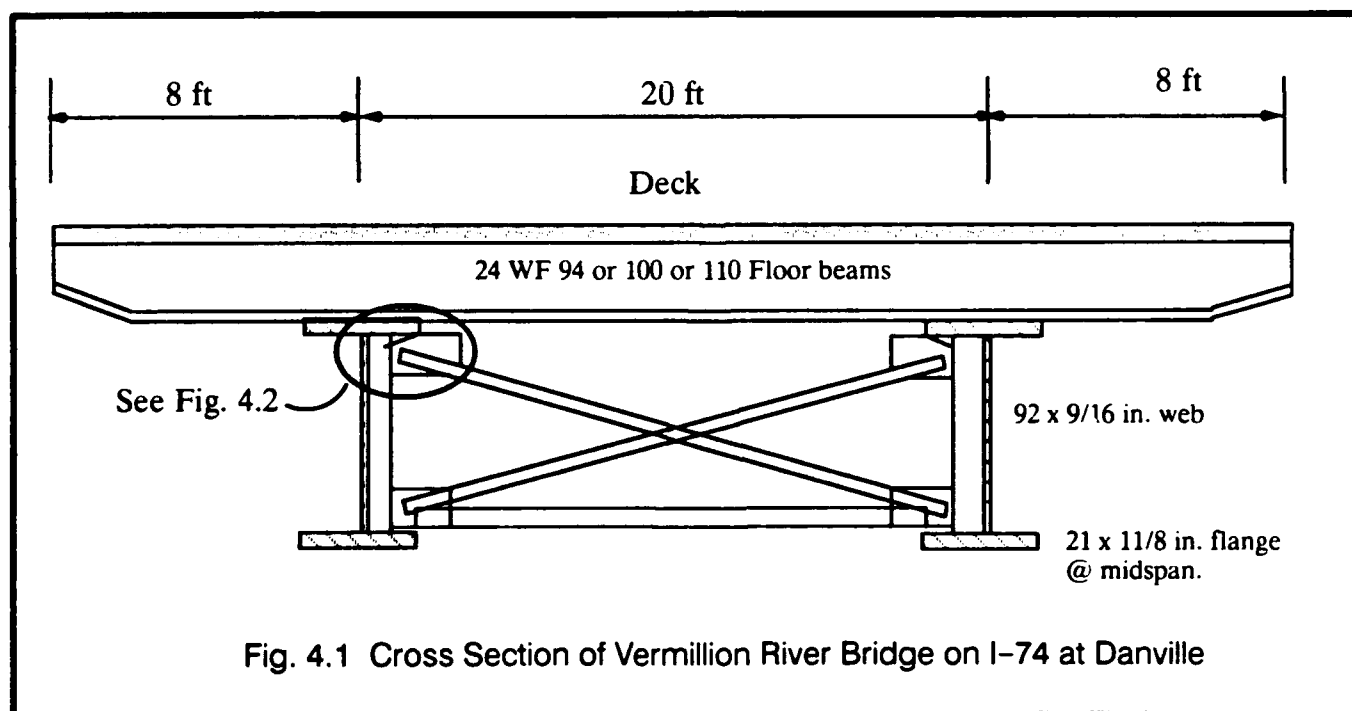
not clear that the separation detail would prevent the development of the cracking seen. It is not proposed as a remedial detail.

CHAPTER 4

CASE 2: THE I-74 BRIDGE OVER VERMILION RIVER AT DANVILLE

4.1 Field Investigation

The second case history bridge is located in Vermilion County on I-74 over the Vermilion River at Danville. The bridge carries eastbound traffic and was constructed in 1963. It is a 5-span continuous two-girder system with a 60 degree skew. The two outer spans are 100 feet long and the three interior spans are each 126 feet long. The floor beams are spaced at 11.5 foot intervals and the main girders are connected every 23 feet by a cross-frame assembly in the vertical plane and also by lower lateral cross-bracing. A typical cross-frame assembly is shown in Fig. 4.1. The trussed cross frame is not fully symmetrical, that is, it lacks an upper chord member. The overhanging length of the floor beams is variable but the 8 ft. dimension is typical.



As reported by the Illinois Department of Transportation, routine inspections have discovered cracks near the copes of some of the plates connecting the cross-framing to the main girders. Investigations have indicated possible poor weld details in these areas. A sketch of the detail found to be cracked is shown in Fig. 4.2. The combination of the severe

cope and the attachment to a gusset on the top flange of the girder represents a very un-favorable geometry.

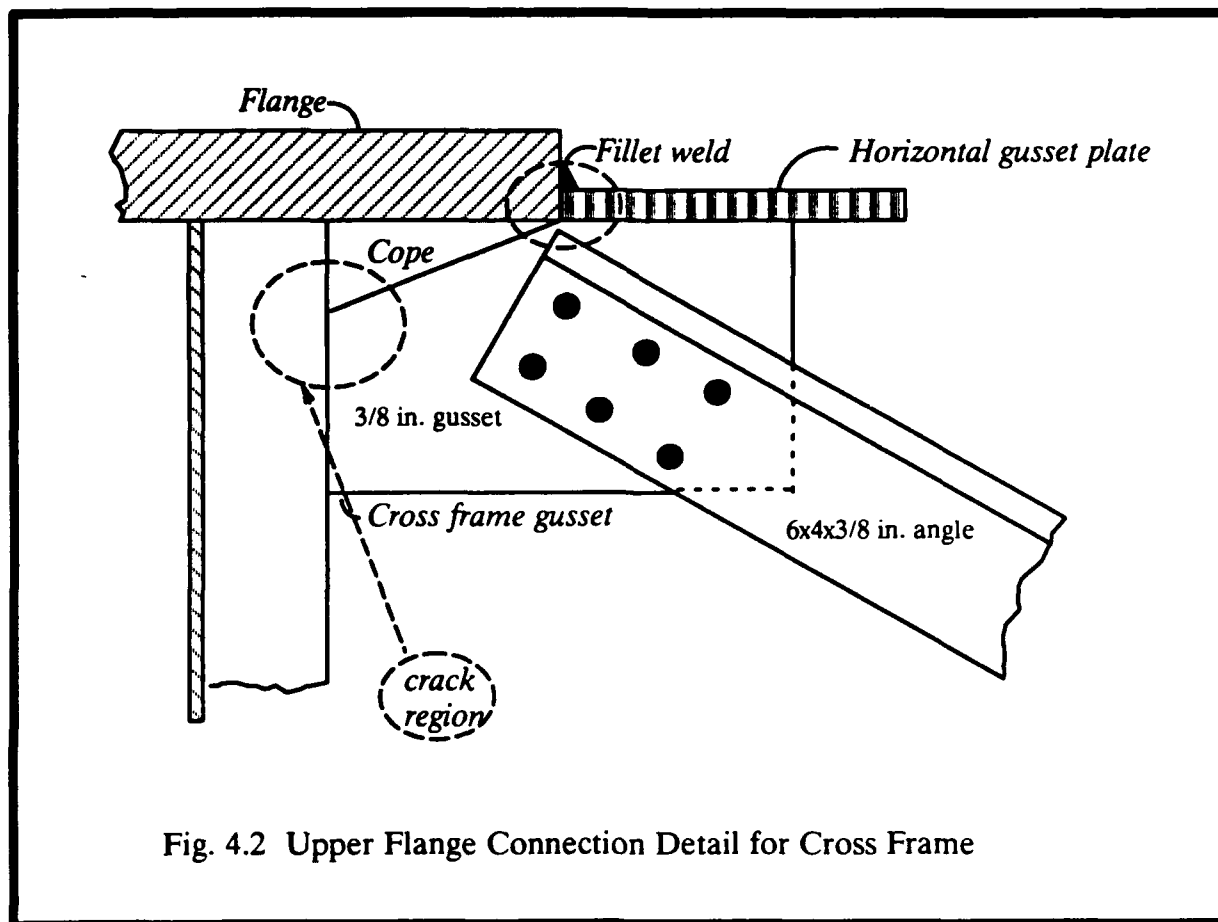


Fig. 4.2 Upper Flange Connection Detail for Cross Frame

The damage in this case history is clearly the result of fatigue at an unfavorable connection detail. The stress concentration at the cope and fillet weld is simply unacceptable as indicated by the cracking observed. The retrofit, given the limited effectiveness of the cross frames in the main structural action of the bridge, was designed to reduce the axial forces in the cross frame diagonals. The option of removing the cross frames is not acceptable since their presence does contribute to the stability of the structural system.

This case study includes:

- Study of the influence of vehicle placement on cross frame response using results from a finite element model of the bridge loaded with a 32 kip axle.
- Analysis of retrofit scheme proposed by I.D.O.T. using neoprene strain-relief devices (Source: C. Hahin, Bureau of Materials and Physical Research) in the cross-bracing to reduce the stress in the connection brackets.

- The determination of the maximum axial force in the cross-frame members both before and after the application in the analytical model of the retrofit proposed by the I.D.O.T

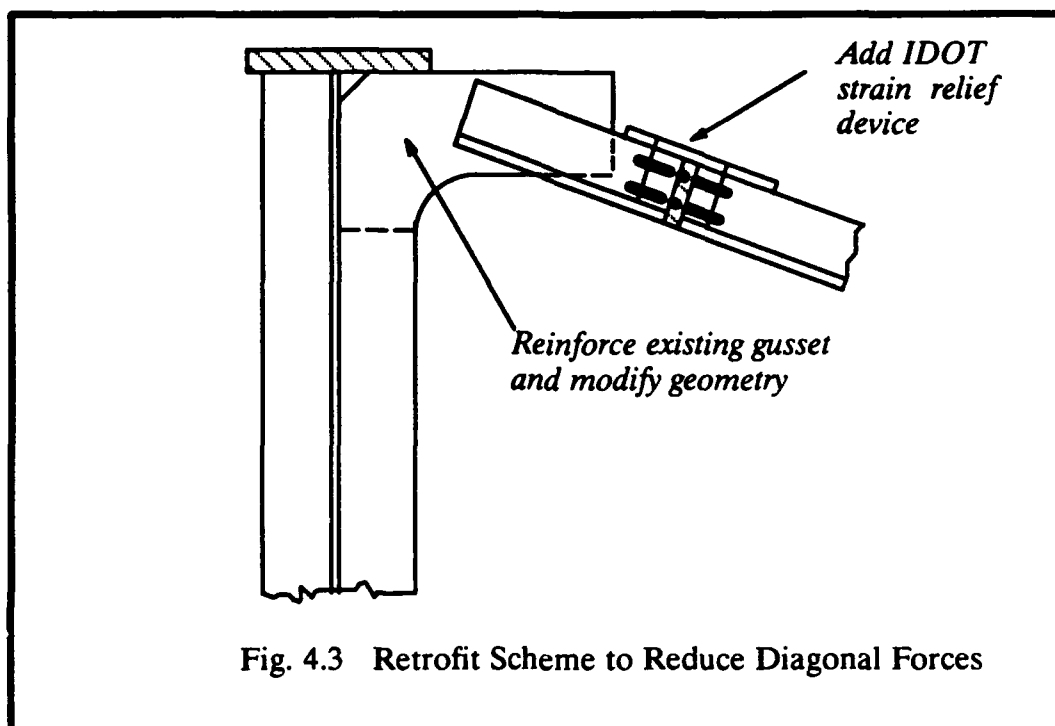


Fig. 4.3 Retrofit Scheme to Reduce Diagonal Forces

4.2 Structural Analysis of Bridge

The I-74 bridge was modeled as a 60 degree skew continuous space frame comprised of three spans of the five, using the finite element modeling program, FINITE. The spans are 126.5 ft each with floor beams spaced 11.5 ft intervals and cross-frame assemblies at 23 ft intervals. Figure 4.4 shows a plan view of the model. The effects of the deck, parapet, lower lateral cross-bracing, and stringers were ignored. It was assumed that the behavior of the cross frame in a given span would be affected mainly by the adjacent spans -- hence, 3-span model results would give a sufficient representation of structural action relative to cross frame behavior.

● support point

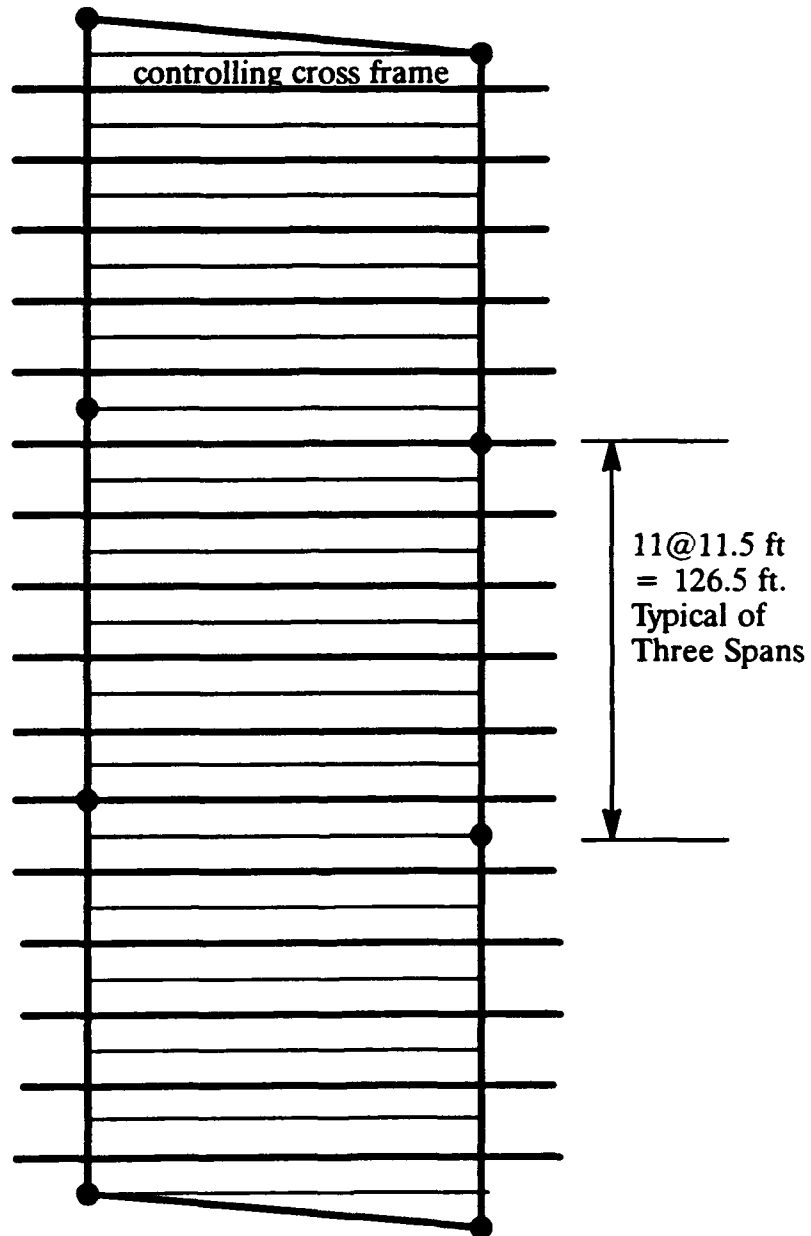
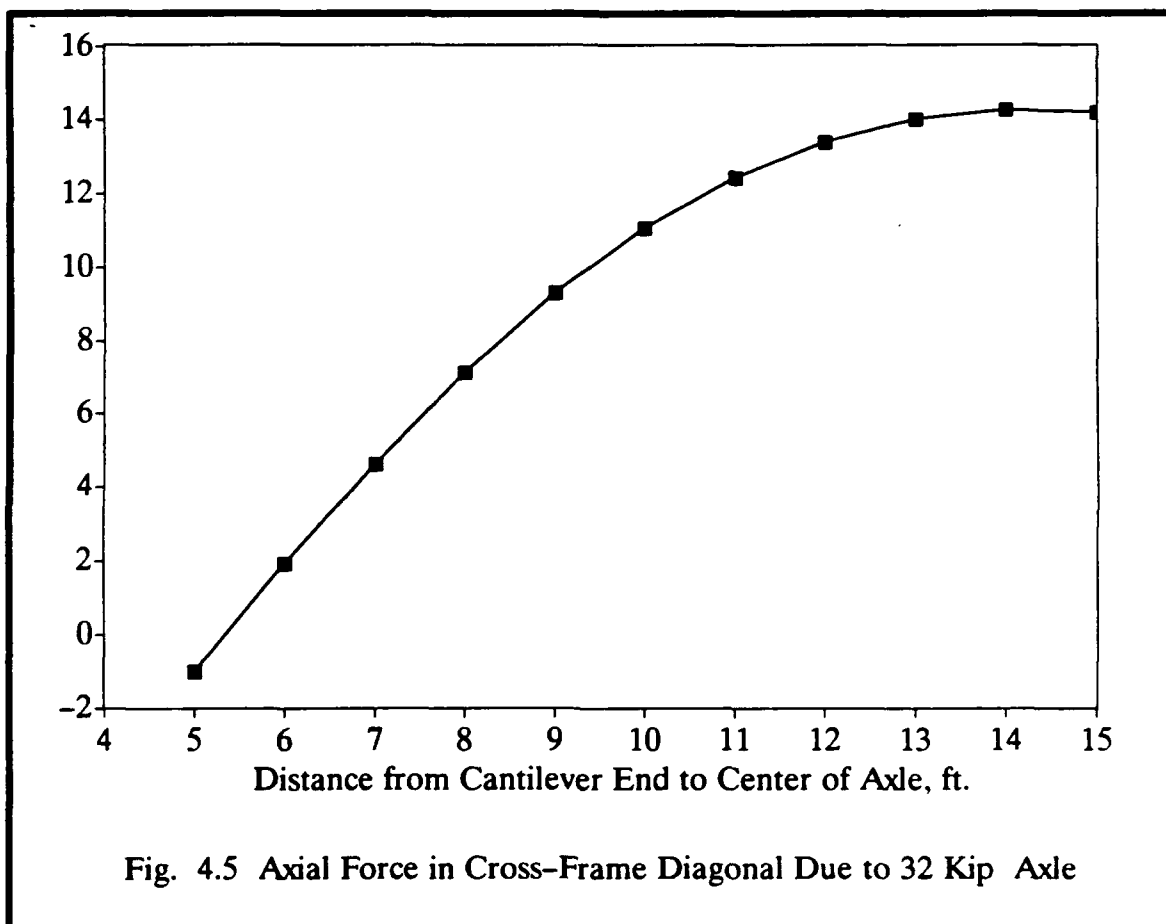


Fig. 4.4 Plan View of Three-Span Grid Model of Bridge

A 32 kip axle was moved longitudinally down the bridge to determine the location of the controlling cross-frame and axle position. The maximum axial force occurred in the first cross-frame from the bridge end when the axle was directly above the cross-frame. Next, the axle was stepped transversely across the corresponding floor beam to determine the maximum axial force in the controlling cross-frame. The resulting maximum axial force was 14.2 kips. A plot of the influence of the transverse location of the axle on the axial force in the cross-bracing is shown in Fig. 4.5.



4.3 Analysis of Retrofit

In a preliminary report by IDOT, a retrofit consisting of neoprene strain-relief devices installed in the cross-bracing was suggested to reduce the stress in the connection brackets. To investigate the results of this retrofit a 1-span 60 degree skew space frame, similar to the 3-span model, was loaded with a 32 kip axle in the position shown below in Figure 4.6.

The comparative effect of the retrofit was tested using an equivalent single span model; it was found that the additional complexity of the multispan model was not needed for this comparison. A 5.25" x 2.75" x 0.75" neoprene member element with a modulus of

elasticity of 5ksi was inserted in each of the L6" x 4" x 3/8" truss cross-braces. The retrofit reduced the axial forces in the cross-bracing approximately 80% to 95%. The resulting member forces can be found in Table 4.1.

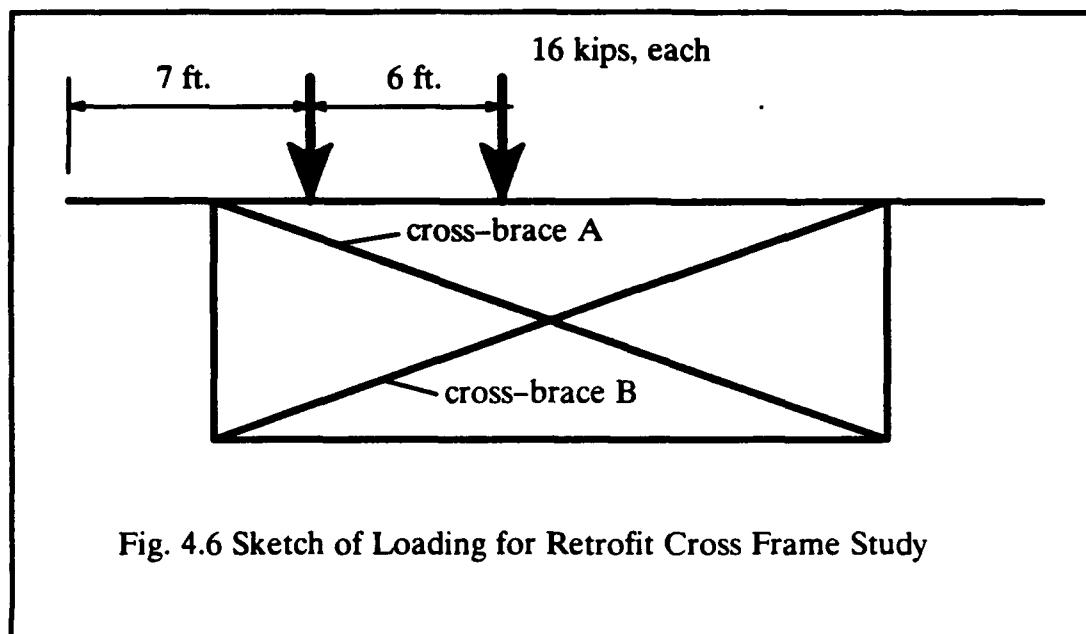


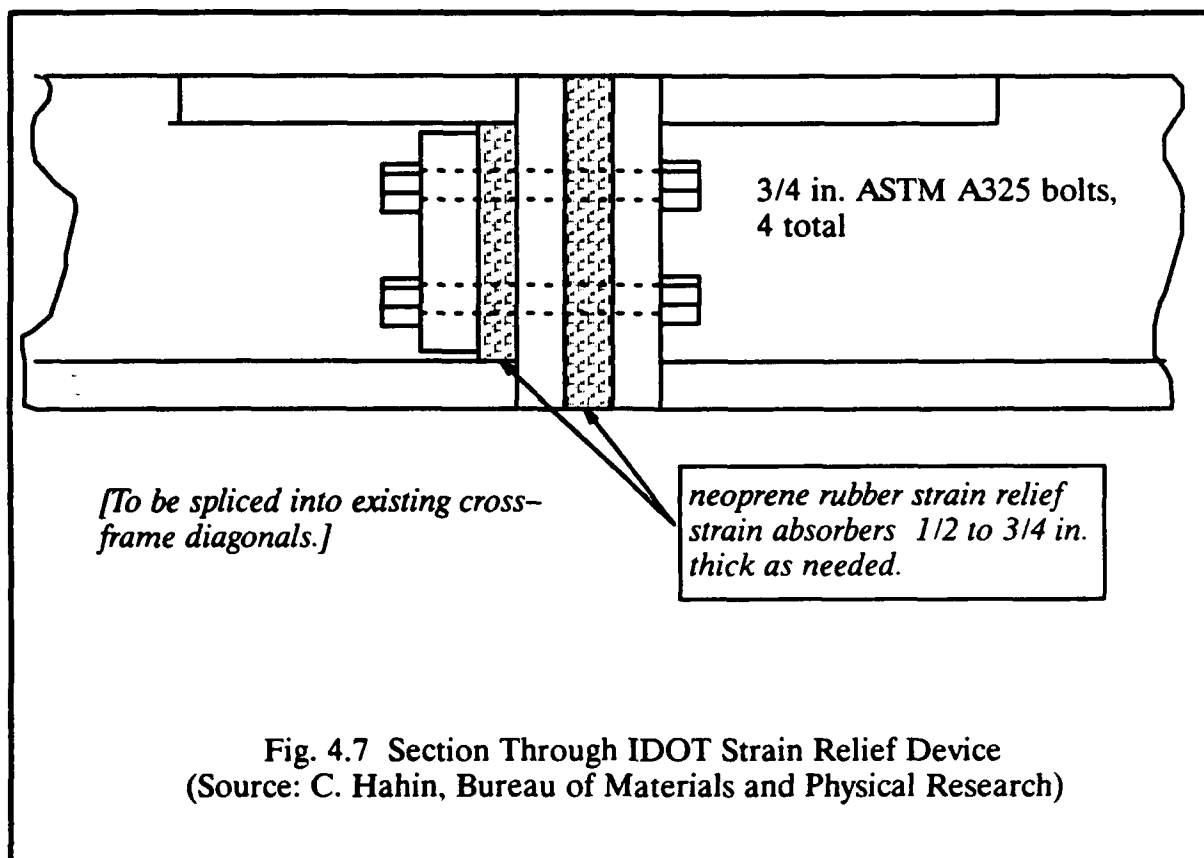
Fig. 4.6 Sketch of Loading for Retrofit Cross Frame Study

Table 4.1 Analytical Test of Proposed Retrofit Design--Axial Forces

x-frame/x-brace	w/o retrofit (kips)	w/retrofit (kips)	retrofit only in x-brace A (kips)
1A	9.6	-0.3	2.6
1B	11.1	1.7	5.7
2A	7.9	-0.3	2.2
2B	9.4	1.7	5.0
3A	7.9	-0.3	2.2
3B	9.4	1.7	5.0

In effect the cross-brace with the retrofit inserted behaves as a soft spring in series with a stiff spring. The total deformation required of the truss element is concentrated in the neoprene member element producing the reduction in member force shown.

It was also of interest to investigate what would happen if one of the strain-relief devices in the cross-frame were installed incorrectly so that the cross-brace would essentially act as a single member. To model this situation, the same 1-span model and loading position



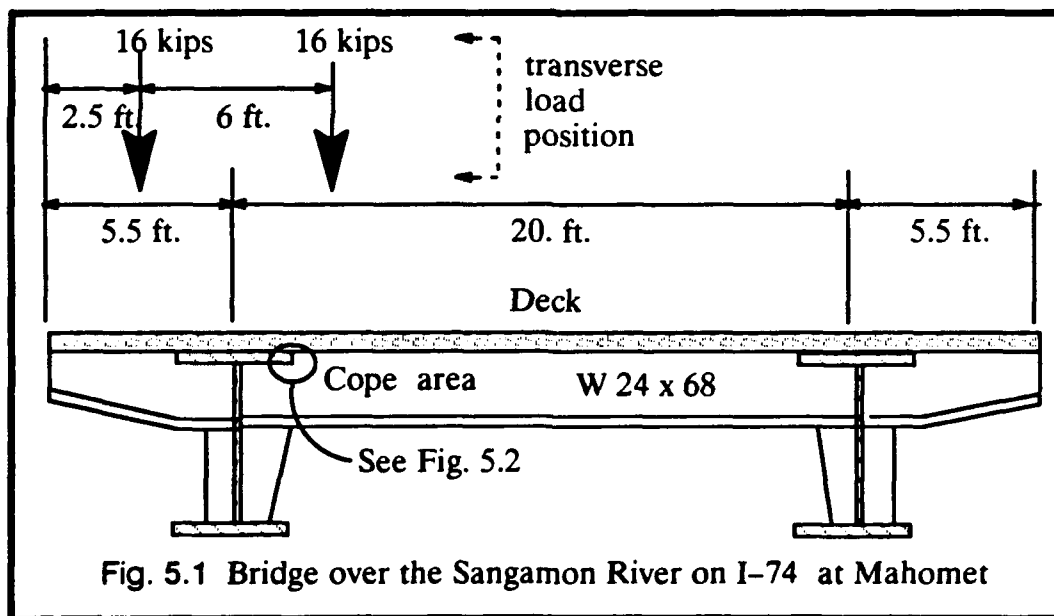
were used, but the neoprene members were removed from cross-brace B in each of the cross-frames. The results are shown in the fourth column of Table 4.1; even with this partial reduction in the stiffness of the cross frame the axial forces are substantially reduced (column two versus column four in Table 4.1.).

CHAPTER 5

CASE 3: MAHOMET BRIDGE ON I-74 OVER THE SANGAMON RIVER

5.1 Field Investigation

The third case history is concerned with the pair of bridges on I-74 over the Sangamon River, east of Mahomet in Champaign County. The east-bound and west-bound bridges were built in 1966. Each bridge is a 3-span continuous two-girder system with side spans of 105.9 feet and a center span of 135 feet. The top flange of the floor beams are coped at the connections to the longitudinal girders to provide clearance for the top flange of the girders. The structural scheme is sketched in Fig. 5.1. Note that the main girders have 17 x 1.5 in. flanges with 45 x 5/8 in. webs.



According to a report by the Illinois Department of Transportation [5], an inspection in December of 1984 revealed cracks at the copes of some of the floor beams, especially at floor beams located 22 to 33 feet from the west side abutment of the east-bound bridge.

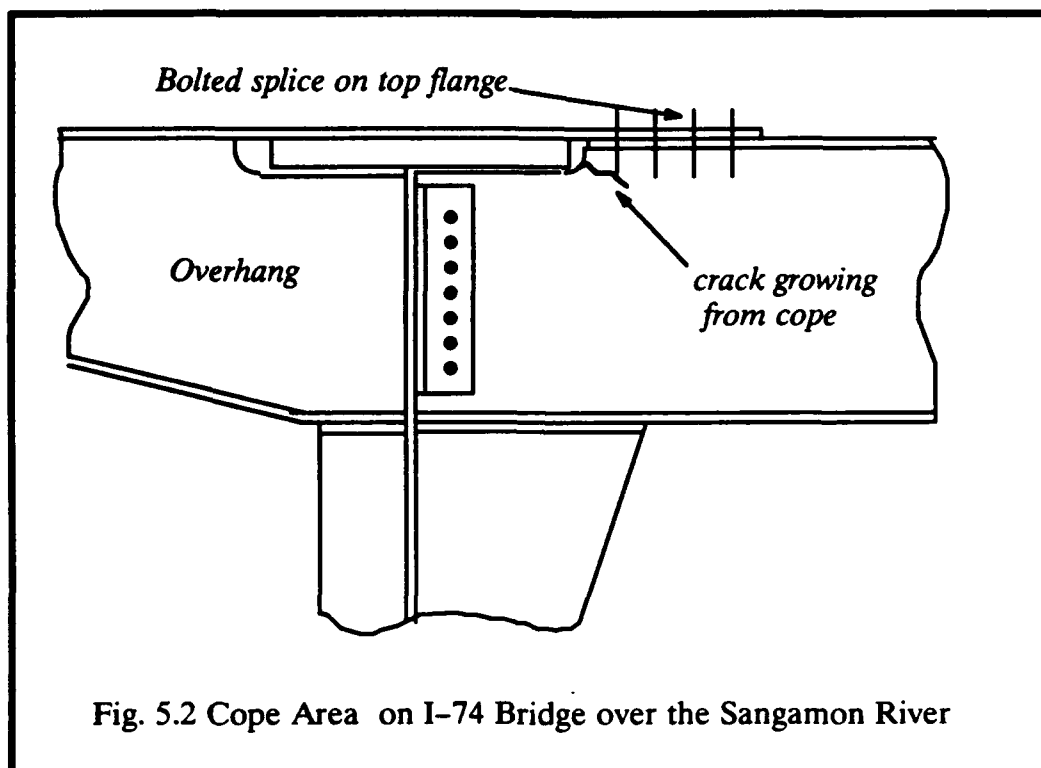


Fig. 5.2 Cope Area on I-74 Bridge over the Sangamon River

Holes were drilled to arrest the cracks as a preliminary retrofit. This retrofit possibility was explored using a finite element analysis with the resulting conclusion that stress would not be suitably reduced. The use of reinforcing plates at the cope was recommended [7].

Field measurements of the strain values around the cope were taken by the IDOT to determine the stress state. A floor beam 22 feet from the west abutment of the east-bound bridge was fitted with a strain-rosette located as close as possible to the cope. The bridge was loaded with a 15 ton truck traveling at 20 mph. The maximum strain state was reached when the truck was near the center of the middle span. According to the IDOT calculations the stress state at the cope when the truck was at the center of the adjacent span is:

$$\sigma_y = 2.65 \quad \sigma_x = 5.33 \quad \tau_{xy} = 2.15$$

All computations and conclusions in this investigation are based on the existing bridge data and field measurements, provided by the IDOT investigation [7]. The analysis undertaken herein for this case will be done in two phases:

- Results for primary member forces from a space frame analysis of the bridge loaded with a 32 kip axle, and

- Results from a finite-element analysis of a section in the region of the cope for a typical floor beam.

The purpose of this step in the study is to try to develop representative models of the entire bridge and of a typical floor beam to determine if an identifiable load effect produced by vehicle passages caused the cracks to develop.

5.2 Structural Analysis of Bridge

The I-74 bridge was modeled as a 3-span continuous space frame using the finite-element modeling program FINITE. The model has exterior spans of 110 feet long and an interior span of 132 feet long. To simplify the model, the longitudinal girders are assumed to have an average depth of 48 inches; however, the girders in the actual bridge have a variable cross-section. The W24 x 68 floor beams are spaced at 11 foot intervals. The cantilevered sections of the floor beams actually have a linearly varying cross-section, but for analysis purposes they were modeled as W24 x 68 sections. The 8.5 inch slab was modeled using 'RFSHELL' elements [8] with a thickness of 8.5 inches, a Poisson's ratio of 0.15, and a Young's modulus of 3000 ksi. The 'RFSHELL' elements are rectangular and provide for both in plane membrane forces and plate bending. Figure 5.1 shows a typical cross-section through the bridge.

A 32 kip axle was stepped transversely across a floor beam located 22 feet from the end support in an outer span until a maximum effect was obtained. The maximum moment occurs when the center of the axle is 5.5 feet from the end of the cantilevered section of the floor beam. The resulting shear and moment at the connection are:

$$M = 280 \text{ kip-in}$$

$$V = 1.61 \text{ kips}$$

Next, the axle was stepped transversely across the middle of the center span. The maximum shear in the floor beam 22 feet from the end support, occurs when the axle is 5.5 feet from the end of the cantilevered section of the floor beam. The axle position is shown also in Fig. 5.1. The moment in the floor beam is close to zero for all transverse axle positions and the shear is:

$$V = 1.52 \text{ kips}$$

The maximum stresses in the space frame model are generated when the axle is directly over the floor beam in question. It is unclear why the maximum stresses measured in the field in the subject floor beam resulted when the truck is near the center of the middle span. The high stresses in the floor beams may be due to differential upward deflection of the longitudinal girders when the bridge is loaded in the center span. Because the bridge has no

skew, it is unknown why the uplift forces would produce a significant differential deflection. This condition did not occur in the analytical model. Since the measured stresses from the field cannot be reproduced in a general space frame model of the full bridge structure, the exact magnitudes of the shear and moments in the floor beam are not determined. However, it is still useful to make a finite-element analysis of the floor beam cope using specified inputs shear and moment which are retained as parameters of the problem.

5.3 Finite-Element Analysis of the Floor Beam Cope

A finite-element mesh consisting of 8-node, quadratic, isoparametric elements ('Q2DISOP'[8]), was used to model a 60 inch long end section of a typical floor beam.

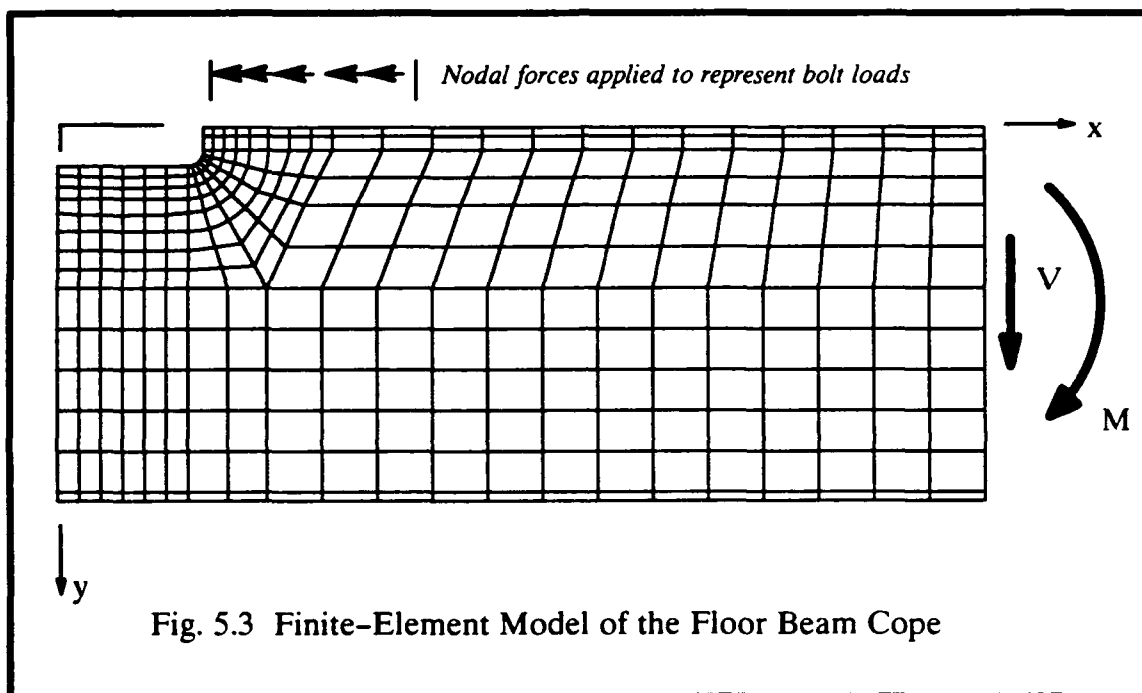


Fig. 5.3 Finite-Element Model of the Floor Beam Cope

The finite-element mesh used to model the floor beam is shown in Figs. 5.3 and 5.4. The beam is fixed on the end which connects to the longitudinal girder and free at the other end. To determine the shear and moment relationship at the free end, the interior floor beam was modelled as a fixed-fixed beam with a vertical displacement release on one end. Displacing the released end causes a constant shear and linear moment in the beam which has the following relationship.

$$M = V(L/2 - x)$$

Where:

L = span length (240 inches)

x = position along beam (60 inches)

These equations yield the following for the applied moment and shear at the free end of the partial floor beam model: $M = 60V$ (kip-in).

The top flange of each cantilevered floor beam in the existing structure extends across the top flange of the longitudinal girders and is connected to the top flange of the coped floor beams with five bolts. To model this condition nodal forces are applied to the top flange of the coped floor beam, with five nodal forces along the top of the flange and five along the corresponding nodes on the bottom.

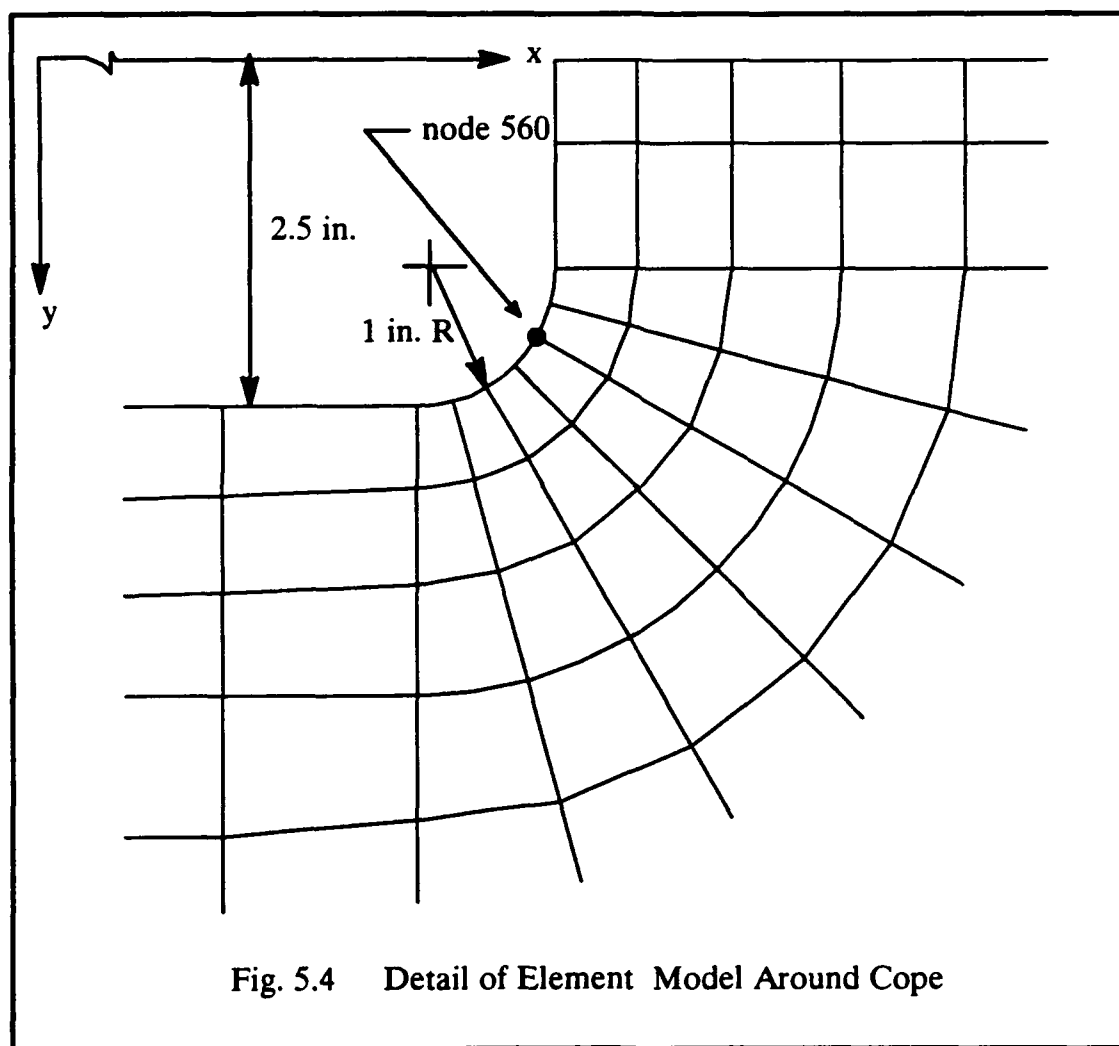


Fig. 5.4 Detail of Element Model Around Cope

Because the magnitudes of the shear, moment, and nodal bolt loads are unknown, unit loads are applied to the floor beam to explore the sensitivity of the problem. The shear and moment forces are represented as a separate loading set from the nodal bolt loads. The stresses at a particular node for each loading condition are multiplied by two separate magnitude factors and superimposed to obtain the stresses at that node. The closest stress

state to the measurements obtained in the field occurs at node 560 which is at the edge of the cope. The corresponding shear, moment, and nodal bolt forces are:

$$V = 1.60 \text{ kips}$$

$$M = 96 \text{ kip-in}$$

$$P = 0.2 \text{ kip (shear force in one bolt)}$$

Figure 5.4 shows a detail of the finite element mesh around the cope. The stresses at node 560 from the finite-element model and the stresses measured in the field are shown in Table 5.1 below. The variability of σ_x , σ_y , and τ_{xy} with position around the cope as defines by the angle, Θ , are shown in Fig. 5.5.

Table 5.1 Comparison of Computed and Measured Stresses
(Stresses Computed at Node 560)

	σ_x (ksi)	σ_y (ksi)	τ_{xy} (ksi)
Finite-Element Model	2.80	5.21	-3.26
Field Measurements	2.65	5.33	-2.15

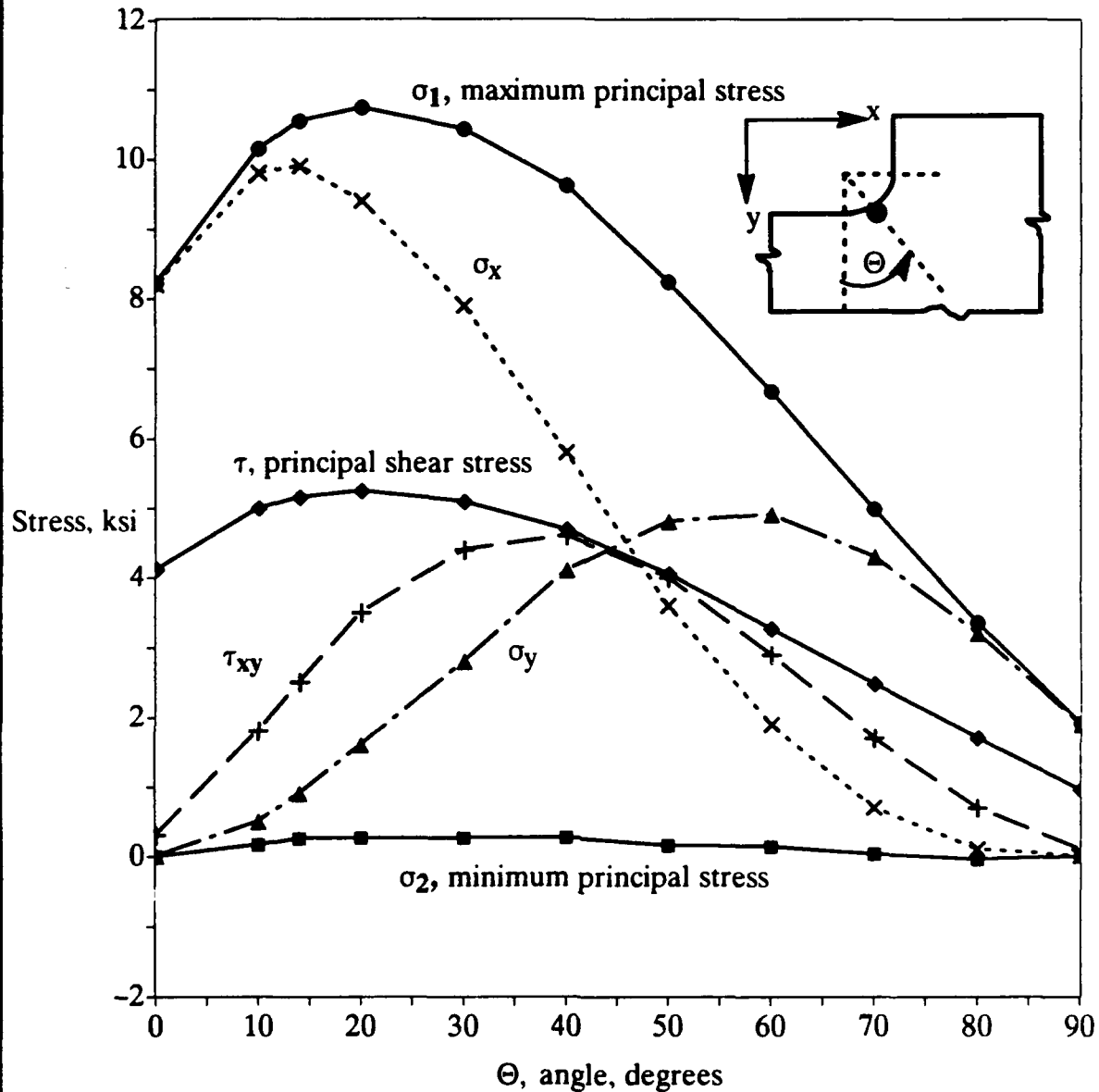


Fig. 5.5 FEM Study of Stresses Adjacent to Beam Cope

The results from the analysis are reasonably good considering that there are an infinite number of possible loading conditions and that a small two-dimensional mesh is being used to model a complex three-dimensional structure. The applied shear in the finite-element model ($V = 1.60$ kips) is very close to the resulting shear in the floor beam when the space

frame model is loaded at the center of the middle span ($V = 1.52$ kips) or at the floor beam ($V = 1.61$ kips). However, the resulting applied moment in the finite-element model is not close to either of the moments for the loading cases in the space frame analysis.

The issues concerning the prediction of stress conditions at the cope raised in this case need additional field testing with more extensive instrumentation to be resolved. This should be accomplished to guide the development of a more complete analytical model for the bridge which would include both frame and deck elements. The results of the present study also indicate some care is needed in assessing the behavior of and modelling of the bolted splice plate on the top flange of the floor beam.

Repairs and modifications of cope details can involve two or more levels of action. First, the simplest level of action is to drill a hole at the end of the crack, removing all visible evidence of the crack tip. This action must be combined with continued regular inspection of the repairs to detect possible re-initiation of the crack. The second level of action involves removing the crack tip with a drilled hole and adding gusset plates on either side of the coped region of the web, fastened with high-strength bolts. The gusset plates will serve to reduce the stresses in the region of the crack to halt further crack propagation.

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary and Conclusions

Floor system cracking problems, the focus of the three case studies of this research study, occur in both structures of recent construction and in older bridges which have been in service for many years. The issues associated with the existence of cracks in floor systems are a subset of more general problems associated with the scheduling of maintenance, rehabilitation and the planning for the replacement of bridge structures. The present study has arisen out of three specific cases of floor system cracks which were not immediately detrimental to the structural integrity of the floor system but which required inspection, monitoring, detailed investigation and eventual remedial action. At the outset these cases of cracking did not seem related to a usual design basis load effects.

The cases forming the central focus of this report are: (1) The I-474 Shade-Lohmann Bridge over the Illinois River south of Peoria., (2) the I-74 Bridge over the Vermillion River at Danville, and (3) The I-74 Bridge over the Sangamon near Mahomet in Champaign County.

These case histories produced evidence of three issues related to cracking:

- Case 1: Cracking due to forces not associated with vertical vehicle loading. That is, evidence is strong that cracking arises from longitudinal load transmitted through the out of plane flexure of the floor beam web in the segment between the connection clip angles and the flange at either end. Calculations show that a very modest induced longitudinal deformations are associated with substantial local stresses at the web to flange junction which is the site of the cracking.
- Case 2: Fatigue failure of a detail with adverse geometry but with forces induced by vehicle loads which clearly account for the damage. A repair detail to reduce member forces has been suggested and is evaluated.
- Case 3: The development of a fatigue crack at a cope detail associated with a reasonable stress state for the damage observed, but with poor correlation with a limited controlled vehicle test and predictions of bridge behavior using a grid model. A more extensive field study with more extensive instrumentation and a more comprehensive analytical model appear need to resolve uncertainties in this case.

The results of the case histories also serve as a guide to the study of the present class of problems and as an exposition on crack propagation problems at other locations in steel bridge structures. The theoretical background for understanding and predicting the propagation of cracks under cyclic stress applications is well documented in the literature and verified by experimental evidence, and has been summarized here in brief. This knowledge must be combined with information on site specific loading conditions, the structural behavior both on a gross level and in detail at the critical "hot spot" where cracking has been detected.

The task of building an analytical model is approached in phases with a global structural model of the bridge in a three-dimensional space frame or grid form used to predict member forces in the floor beam element near the crack site. With these forces as input a more refined finite element of the crack region is used.

A commentary on methodology is presented.

6.2 On Methodology

The case histories described herein are specialized and do not represent the full range of cracking and fatigue problems seen in steel bridges, for example as has been compiled under the study conducted at the Center for Advanced Technology for Large Structural Systems (ATLSS), Lehigh University, by Fisher, Yen, Wang and Demers [14, 15, 16]. Fisher *et al* also add a useful summary of basic data and theory with excellent illustrations. Demers and Fisher [14] have surveyed cases of local cracking in steel bridges over the period 1981-1988 and have a variety of classifications and cases bearing upon the present project; the extent of this relationship can be seen in Table 6.1, below.

Table 6.1 ATLSS Survey of Cracking in Bridges -- Cases Related to Present Study

Classification of Cracking Problem	No. of Cases	Table in Ref. 14
Coped Members	22	25
Diaphragm Connection Plates (web gaps cited)	15	25
Connecting End Angles on Webs	7	7
Web Gap -- Web to Gusset Details	13	13
Flange-Gusset Plate Connection	10	5
Cover Plate Splices	5	8
Welded Cover Plate Termination	5	4

The following statements on methodology will be focussed on the floor system problems encountered. A general pattern of tasks emerges from the three cases presented in

this report. Most if not all of the following tasks, common to all fracture control plans, represent the general methodology for handling a review of a floor system cracking problem:

- Information Needs and Data Gathering. The most important initial need is for a careful inspection of the structure to assess the extent of visible cracking.
- General Analytical Tasks
- Site Specific Crack Initiation and Propagation Studies
- Catastrophic Failure Assessment
- Field Testing and Assessment
- Assessment of Repair Details
- A Systematic Program of Inspection

The severity of the cracking problem must determine the degree to which all or part of the above tasks are undertaken. If no action is taken then sites of visible cracking must be subject to ongoing periodic inspection. A full investigation might not be cost effective, or might exceed the cost of simply using a known and reliable repair detail without extensive prior study; again, if such is the chosen course of action then a program of periodic inspection remains important.

The general analytical tasks must be approached with an appropriate level of refinement to match the task at hand. Full modelling of the entire structure is not feasible, nor always necessary. Good estimates of major member forces near the crack site are needed and then are used as input to a refined local model of the crack site or of the proposed repair detail at that site. Thus it is true that for structures where inspections have not as yet revealed cracking, good judgement and reference to other cracking surveys must be used to select "hot spots" to review. In the study of the repair detail in Case 2, the repair detail has the effect of reducing the major member forces as well as reducing the stress intensity at the crack site.

A possible path for the investigative process is diagrammed in Fig. 6.1. One can envision a set of outcomes which may involve a decision to do one of the following:

1. repair using a detail which has been evaluated by the methodology outlined,
2. do not repair, but establish an inspection schedule and monitor crack behavior,
3. establish that no significant problem exists and monitor as any other structure, or,

4. establish that a total replacement of the floor system is needed, with a modified design -- the most drastic outcome.

In the decision making process, the use of calculations of remaining life in most instances should be taken as base line for relative not absolute measures of improved life -- particularly in the assessment of repair details. Although there are uncertainties in fatigue or crack propagation behavior, the variability in the loadings and the uncertainties in making measurements or calculations of the stress history at the fatigue critical location are the greatest hindrance to the forecasting of an absolute fatigue life. Usually the lack of data for a life-long load history is the most important factor that prevents a greater refinement in fatigue life estimates. That is, both traffic volume data and correlated information on vehicle GVW and dimensions are unavailable for every bridge.

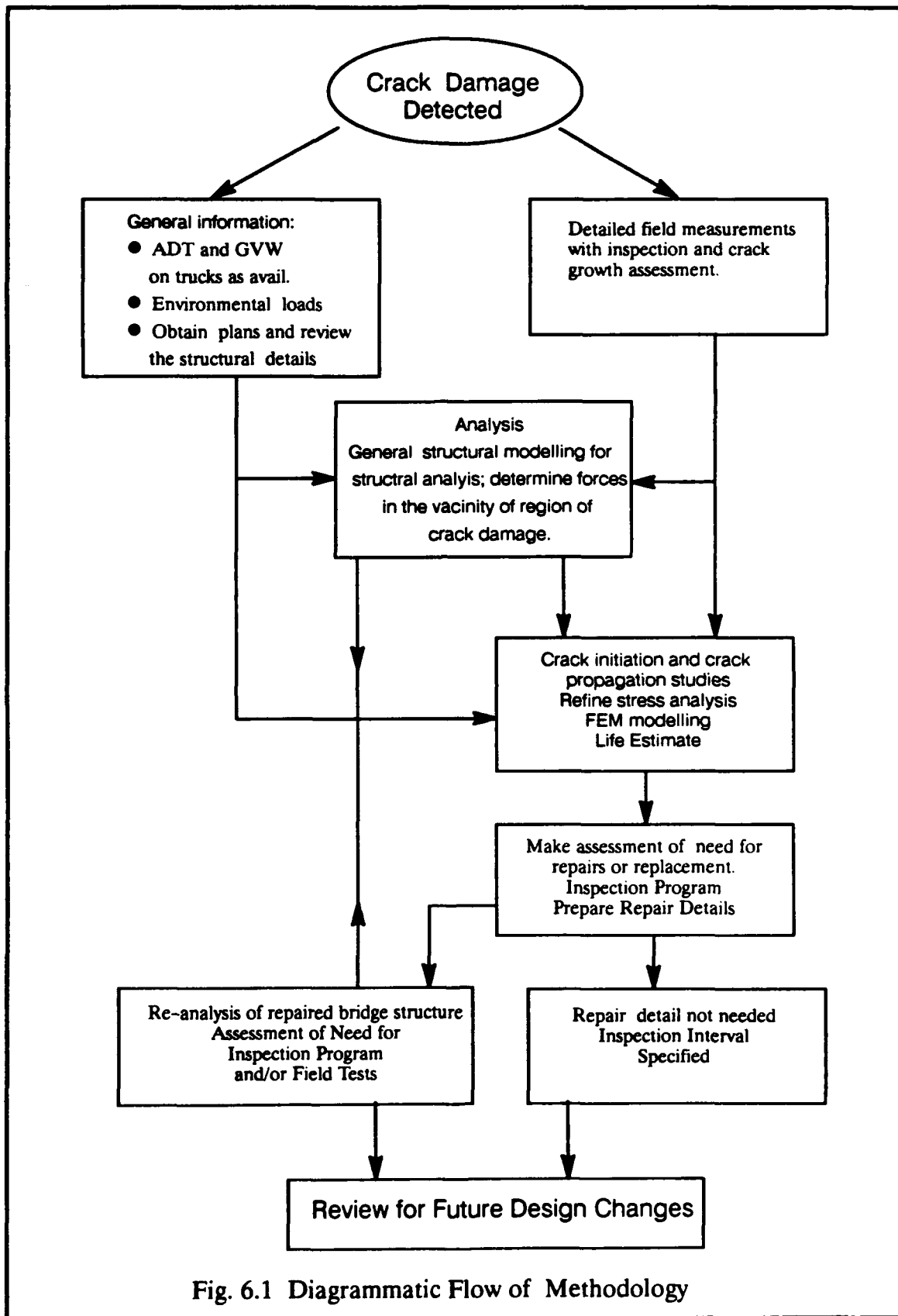
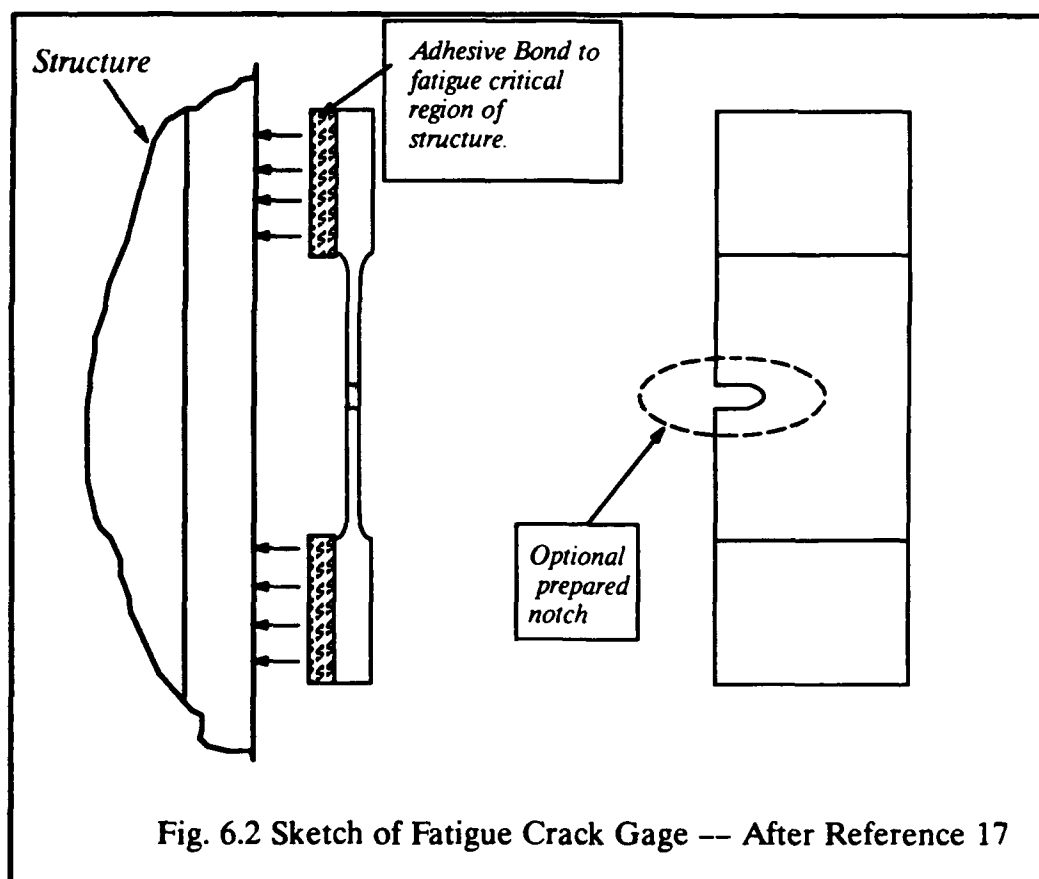


Fig. 6.1 Diagrammatic Flow of Methodology

Follow-up studies after repairs have been made or corrective action such as holes drilled at crack tips should include careful monitoring for crack re-initiation or growth of existing cracks. So-called fatigue gages [17] can be used to assess the potential for new crack formation or additional damage. Nmai and Bowman in Ref. 17 provide a discussion of the scaling factors needed to adjust the gage dimensions and notch size to a given situation. The gage is sketched in Fig. 6.2 and may either have a prepared notch or be unnotched. The presence of the notch will permit the modelling of a pre-set degree of damage in an assessment of remaining life. It is a small flat plate coupon that is bonded with adhesive to the structure in the vicinity of the fatigue critical location. Thus it will experience similar stress variations. The use of such gages originated in the aircraft industry.



6.3 Future Work

Missing from the scope of the present study was the ability to do a sustained study of a case where the application of the remedial measure and follow-up field study of the response could be accomplished. Since field work is costly, it should be considered only where cost effective, for example:

- For a repetitive detail to be applied extensively on many structures.
Example: a redesign of trussed bracing details.
- For a major repair where it is essential to monitor performance and check for possible re-development of damage.

Any extensive field program should be on structures with a reasonable life expectancy not limited by other factors such as deterioration due to extensive corrosion or geometric obsolescence.

Longitudinal forces in bridge deck systems appear to be handled in a variety of ways, perhaps without adequate attention of the actual load path. The present study has added another case of damage (the Shade Lohmann Bridge) attributable to this source of adverse loading. Field studies of this behavior combined with analytical studies could be helpful in revising design criteria.

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